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Rehabilitation Strategies for Highway Pavements

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Table of Contents

Table of Contents, continued

Table of Contents, continued

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Rehabilitation Strategies for Highway Pavements

Summary of Findings

NCHRP Project 1-38 was conducted to develop a process for selection of appropriate rehabilitation strategies for the ranges of pavement types and conditions found in the United States. A review of the pavement rehabilitation practices of State DOTs, and the literature available on pavement evaluation, rehabilitation techniques, and selection of rehabilitation strategies, was conducted for this project.

Although all State DOT agencies are engaged in pavement rehabilitation, fairly few of them have any more than the most simple and general guidelines for selection of rehabilitation strategies. The rehabilitation strategy selection procedures used by the various highway agencies differ in their details, but typically consist of: (1) data collection, (2) pavement evaluation, (3) selection of rehabilitation techniques, (4) formation of rehabilitation strategies, (5) life-cycle cost analysis, and (6) selection one pavement rehabilitation strategy from among the alternatives considered. This report provides a step-by-step process and guidelines for each of these activities.

Trigger values are suggested for key condition levels (asphalt pavement cracking, concrete pavement cracking and faulting, composite pavement reflection crack deterioration) at which a pavement is generally considered to need a structural improvement. Similarly, trigger values are suggested for key condition levels (asphalt pavement rutting, concrete pavement faulting, serviceability in all pavement types) at which a pavement is generally considered to need a functional improvement.

Overlay rehabilitation strategies require that the overlay thickness be determined. Several overlay design procedures are available; this report does not provide details about the use of these procedures. The report also does not provide performance prediction models for other types of rehabilitation. However, general estimates of the service life ranges for several different types of rehabilitation strategies have been provided. The ranges are intended to represent the service lives that may reasonably be expected of the different rehabilitation techniques; service life estimates used by specific State DOTs or other agencies are cited where possible for comparison. However, it is emphasized that the performance of a rehabilitation strategy depends on many factors, among which are the condition of the existing pavement, the amount of pretreatment repair done, the rehabilitation design, the quality of construction, the materials used, the climate, and the traffic level.

Drainage improvement options are presented with the caveat that it is difficult to predict what effects, either positive or negative, drainage improvement efforts may have on the performance of a rehabilitated pavement. Very little research is available to demonstrate how retroffitted subdrainage influences in-service pavement performance.

The report provides detailed guidance on how to conduct a life-cycle cost analysis of rehabilitation strategy alternatives. It does not provide default unit costs for the different items which may enter into a life-cycle cost analysis.

Despite the enormous amount of funding dedicated to pavement rehabilitation in the United States every year, the pavement field’s ability to predict the performance of different rehabilitation techniques remains very limited. A great deal has been written about how rehabilitation techniques should be constructed and what materials should be used, but relatively little useful research has been done into how long and how well these different rehabilitation techniques perform. This report provides a step-by-step process for project-level evaluation of pavements in need of rehabilitation, selection of rehabilitation techniques believed to be appropriate, and formation of rehabilitation strategies expected to be feasible and cost-effective. As the pavement field’s ability to predict rehabilitation performance improves, this process may be further refined and customized to the needs of individual State agencies.

Chapter 1

Introduction and Approach

This report provides a step-by-step process and guidance for project-level evaluation and rehabilitation strategy selection for highway pavements. The identification of an individual pavement as one which needs rehabilitation in the near future, and therefore warrants this kind of project-level evaluation effort, is a network-level pavement management activity which precedes a project-level analysis. This work was conducted under NCHRP Project 1-38.

Definitions

Pavement rehabilitation is defined as a structural or functional enhancement of a pavement which produces a substantial extension in service life, by substantially improving pavement condition and ride quality. **Pavement maintenance** activities, on the other hand, are those treatments that preserve pavement pavement condition, safety, and ride quality, and therefore aid a pavement in achieving its design life. Pavement maintenance activities are <u>not</u> addressed in this report.

Individual rehabilitation treatments are often categorized as belonging to one of the “4 R’s” – restoration, resurfacing, recycling, or reconstruction. There are some problems with trying to fit each rehabilitation treatment into one of these four major categories. For example, some treatments may be done as part of a restoration effort or as part of a resurfacing effort. The 4 R’s are good descriptors of the type of rehabilitation effort most appropriate at a given point in a pavement’s life, but are less useful as a classification scheme for rehabilitation treatments than as. Each of the four types of rehabilitation is defined below.

Restoration is a set of one or more activities that repair existing distress and significantly increase the serviceability (and therefore, the remaining service life) of the pavement, without substantially increasing the structural capacity of the pavement.

Resurfacing may be either of the following:

(a) A **structural overlay**, which significantly extends the remaining service life by increasing the structural capacity and serviceability of the pavement, usually in combination with preoverlay repair and/or recycling. A structural overlay also corrects any functional deficiencies present.

(b) A **functional overlay**, which significantly extends the service life by correcting functional deficiencies, but which does not significantly increase the structural capacity of the pavement.

Recycling is the process of removing pavement materials for reuse in resurfacing or reconstructing a pavement (or constructing some other pavement). For asphalt pavements, this process may range from in-place recyling of the surface layer, to recycling material from all pavement layers through a hot mix plant. For concrete pavements, recycling involves removal and crushing for reuse as aggregate, either in the reconstruction of the pavement or for surface, base, or subbase layers in other pavement construction. Recycling of asphalt-overlaid concrete pavement may be either surface recycling or removal and recycling of both asphalt and concrete. In this case, the asphalt and concrete layers are removed and recycled separately.

Reconstruction is the removal and replacement of all asphalt and concrete layers, and often the base and subbase layers, in combination with remediation of the subgrade and drainage, and possible geometric changes. Due to its high cost, reconstruction is rarely done solely on the basis of pavement condition. Other circumstances, such as obsolete geometrics, capacity improvement needs, and/or alignment changes, are often involved in the decision to reconstruct a pavement.

Pavement Types Addressed

This report addresses rehabilitation of highway pavements; it does not address rehabilitation of unpaved or surface-treated low-volume roads, nor rehabilitation of urban streets, although many of the concepts and techniques are applicable to these types of facilities. The pavement types addressed in this report are briefly described below.

Asphalt concrete pavement is also sometimes referred to as asphalt pavement or flexible pavement. **Asphalt pavement on untreated or treated base** has a hot-mixed asphalt concrete surface, usually over a base layer which may be either untreated or treated granular material, and possibly a subbase layer (usually untreated). **Full-depth asphalt concrete pavements** are those in which all layers contain an asphaltic binder. The asphalt concrete layers (which may be of different gradations and asphalt cement contents) are constructed directly on the prepared subgrade.

Portland cement concrete pavement is also sometimes referred to as concrete pavement or rigid pavement. **Jointed plain concrete pavement (JPCP)** has transverse joints typically spaced less than about 20 ft apart, and no reinforcing steel is provided in the slabs. It may have steel dowel bars across transverse joints, and steel tiebars across longitudinal joints. **Jointed reinforced concrete pavement (JRCP)** has transverse joints typically spaced more than 20 ft apart. The reinforcement (welded wire fabric or deformed steel bars) comprises about 0.15 to 0.25 percent of the cross-sectional area of the slab. Due to its longer joint spacing, jointed reinforced concrete pavement is expected to develop midslab cracks. The purpose of the steel reinforcement is to keep these cracks tight. Transverse joints are typically doweled in jointed reinforced concrete pavement. **Continuously reinforced concrete pavement (CRCP)** does not have transverse joints, other than the transverse construction joints placed at the end of each day's paving and at abutting pavement ends and bridges. Continuously reinforced concrete pavements have a considerably higher steel content than jointed reinforced concrete pavements – typically 0.6 to 0.8 percent of the cross-sectional area. The purposes of the longitudinal steel are to control the spacing of cracks resulting from drying shrinkage and temperature changes and to keep these cracks tight. Transverse reinforcing steel is often used to support the longitudinal steel during construction and to control any random longitudinal cracks which may develop. All three types of concrete pavements are usually constructed on a layer of untreated or treated granular material, commonly referred to as the base layer. In some cases, a lower-quality gravel is used to separate the base from the subgrade. This layer is commonly referred to as the subbase.

Any of the above types of concrete pavement which has an asphalt concrete surface is an **asphalt-overlaid concrete (AC/PCC) pavement**. Most asphalt-overlaid concrete pavements in service were originally constructed as bare concrete pavements, and later resurfaced with an asphalt overlay. Asphalt-overlaid concrete pavement is sometimes called **composite pavement**, although this term tends to imply new construction as an asphalt-surfaced concrete pavement, rather than a rehabilitated concrete pavement. Concrete pavements with existing asphalt overlays are treated in this report as a third major pavement type, because this pavement type makes up a substantial portion of the high-volume highway mileage of the United States, and because some evaluation and rehabilitation strategy selection for this type of pavement differ in some respects from evaluation and rehabilitation strategy selection for either asphalt pavements or concrete pavements.

Approach

The information presented in this report was developed after a review of the pavement rehabilitation practices of State DOTs and the available literature on pavement evaluation, rehabilitation techniques, and selection of rehabilitation strategies.

The rehabilitation strategy selection process used by different highway agencies differ in their details, but typically consist of the following principal activities:

- **Data collection:** Gathering all of the information necessary to conduct an evaluation of the pavement's present condition and its rehabilitation needs.
- **Pavement evaluation:** Assessing the current condition of the pavement, identifying the key types of deterioration present, identifying deficiencies that must be addressed by rehabilitation, and identifying uniform sections for rehabilitation and design over the project length.
- **Selection of rehabilitation techniques:** Identifying candidate rehabilitation techniques which are best suited to the correction of existing distress and achievement of desired improvements in the structural capacity, functional adequacy, and drainage adequacy of the pavement.
- **Formation of rehabilitation strategies:** Combining individual rehabilitation techniques into one or more rehabilitation strategy alternatives, developed in sufficient detail that the performance and costs of each may be confidently estimated.
- **Life-cycle cost analysis:** Comparing the monetary costs and benefits of the different rehabilitation strategy alternatives over a common analysis period.
- **Selection of rehabilitation strategy:** Considering monetary factors and nonmonetary factors together in selecting one pavement rehabilitation strategy from among the alternatives considered.

A detailed step-by-step process for identifying and comparing one or more appropriate rehabilitation strategies is described in this report. The rehabilitation process is illustrated in Figure 1. The results of the research conducted in this project are presented in this report in the following sequence:

- Chapter 2 – Guidelines for project-level data collection.
- Chapter 3 – Guidelines for structural, functional, and drainage evaluation.
- Chapter 4 – Guidelines for selection of appropriate rehabilitation techniques.

- Chapter 5 – Guidelines for combination of individual rehabilitation techniques into feasible rehabilitation strategy alternatives.
- Chapter 6 – Guidelines for life-cycle cost analysis of rehabilitation strategy alternatives.
- Chapter 7 – Guidelines for consideration of monetary and nonmonetary factors together in the final selection of a pavement rehabilitation strategy.
- Chapter 8 – Summary and conclusions.
- Appendix A – Descriptions of the types of pavement distress and their causes.
- Appendix B – Descriptions of the appropriate use, limitations, concurrent work, materials, design, construction, and performance of the techniques most widely used in pavement rehabilitation.
- Appendix C – A detailed example illustrating the six steps in project-level pavement evaluation and rehabilitation strategy selection.

Overlay rehabilitation strategies require that the overlay thickness be determined. Several overlay design procedures are available; this report does not provide details about the use of these procedures. The report also does not provide performance prediction models for other types of rehabilitation. However, general estimates of the service life ranges for several different types of rehabilitation strategies have been provided. The ranges are intended to represent the service lives that may reasonably be expected of the different rehabilitation techniques; service life estimates used by specific State DOTs or other agencies are cited where possible for comparison. Drainage improvement options are presented with the caveat that it is difficult to predict what effects, either positive or negative, drainage improvement efforts may have on the performance of a rehabilitated pavement. Very little research is available to demonstrate how retroffitted subdrainage influences in-service pavement performance.

The report provides detailed guidance on how to conduct a life-cycle cost analysis of rehabilitation strategy alternatives; it does not provide default unit costs for the different items which may enter into a life-cycle cost analysis.

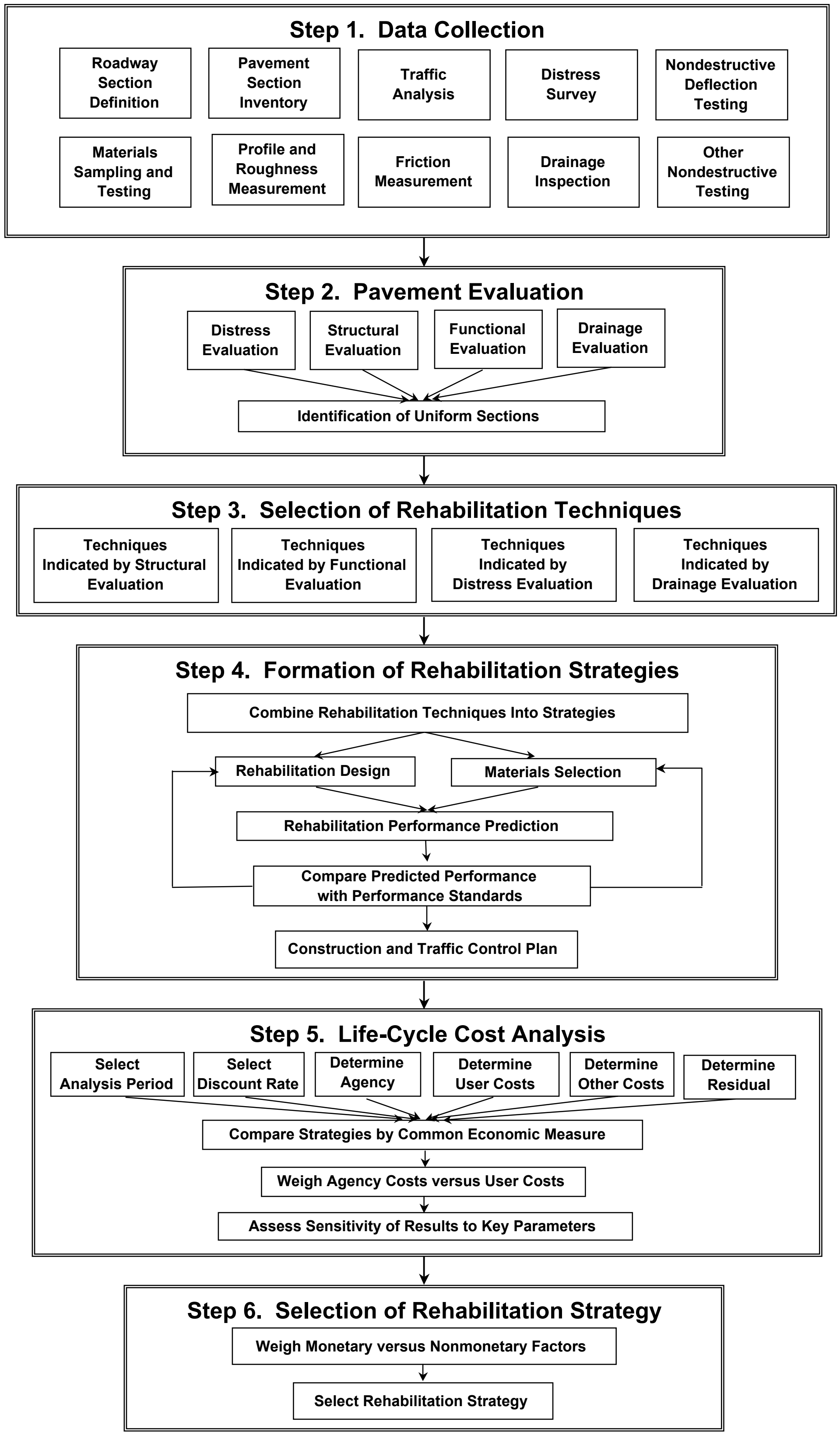

Figure1. Pavement rehabilitation process

Chapter 2

Guidelines for Project-Level Data Collection

The purpose of project-level data collection is to gather all of the information necessary to conduct an evaluation of the pavement´s present condition and rehabilitation needs, develop one or more rehabilitation strategies, predict the performance of each strategy, and estimate the cost of each strategy.

Roadway Section Definition

This involves identifying the location of the project by route name or number, direction, county, nearby city or town, milepost limits, and/or station limits – all information that will be needed to locate the project and estimate rehabilitation costs over its length. Information such as the locations of bridges, underpasses, and interchanges, station equations, etc., should also be noted.

Pavement Section Inventory

This involves examining pavement management files, construction records, and reports from past evaluation and rehabilitation activities for the purpose of determining the pavement type, pavement age, pavement layer materials and thicknesses, number of lanes, widths of lanes and shoulders, predominant subgrade soil type, and subdrainage features.

Traffic Analysis

The current traffic volumes and axle loadings and anticipated traffic growth rates should be determined. With this information, traffic volumes and axle loadings may be forecasted for the design traffic lane (usually the outer lane in one direction) over whatever design periods are later selected for the rehabilitation strategy alternatives considered.

For the purposes of pavement rehabilitation strategy selection, the current and projected future traffic should be characterized in terms of whatever traffic input is used in the resurfacing and reconstruction design procedures used by the agency. In the 1993 AASHTO[1,2] methodology, which is used by many State DOTs, the mixture of anticipated axle loads is expressed in terms of an equivalent number of 18-kip single-axle loads (ESALs). The Asphalt Institute procedures for asphalt pavement design[3] and overlay design[4] also use ESALs as the traffic input. The

Portland Cement Association procedures for concrete pavement design[5] and concrete overlay design[6] use the axle load data directly.

Current ESALs may be estimated in a few different ways, depending on the type of data available. The method used should be the one that will provide the highest level of precision consistent with the available data. Listed in descending order of precision, the alternatives include:

- Using site-specific truck axle distributions, truck axle volumes, and factors for ESALs per axle, i.e., load equivalency factors. Load equivalency factors are a function not only of axle type and weight, but also pavement structure (slab thickness or Structural Number) and design terminal serviceability.
- Using truck distributions, truck volumes, and truck factors (average ESALs per truck). In the absence of site-specific truck distribution data, it is possible to use truck distribution data from other roadways with similar truck usage characteristics.
- Using average daily traffic, percent trucks, and an overall average truck factor. This information may be very approximate, but should also be fairly readily available to a State DOT.

Additional information on traffic data collection is provided in the FHWA *Traffic Monitoring Guide*,[7] NCHRP Synthesis 130 *on Traffic Data Collection and Analysis: Methods and Procedures*,[8] and NCHRP Synthesis 124 on *Use of Weigh-in-Motion Systems for Data Collection and Enforcement*.[9]

Equivalent Flexible and Rigid ESALs

A given stream of truck axle types and weights may be represented in terms of both rigid pavement ESALs and flexible pavement ESALs. When more than one rehabilitation strategy is being considered, it may be necessary to calculate both flexible and rigid pavement ESALs (e.g., when both concrete and asphalt overlay alternatives are being considered for an asphalt pavement). The following recommendations are made for the appropriate type of ESALs to be used for different rehabilitation options:

Use flexible pavement ESALs for:

- Nonoverlay rehabilitation of asphalt pavements,
- Asphalt overlay rehabilitation of asphalt pavements,
- Asphalt overlay rehabilitation of fractured concrete pavements,
- Reconstruction in asphalt of any existing pavement type.

Use rigid pavement ESALs for:

- Nonoverlay rehabilitation of concrete pavements,
- Nonoverlay rehabilitation of existing asphalt-overlaid concrete pavements,
- Asphalt overlay rehabilitation of nonfractured concrete pavements,
- Concrete overlay rehabilitation of asphalt pavements,
- Concrete overlay rehabilitation of concrete pavements,
- Concrete overlay rehabilitation of existing asphalt-overlaid concrete pavements,
- Reconstruction in concrete of any existing pavement type.

Guidelines for calculation of ESALs are provided in Appendix D of the 1993 AASHTO Guide.[1] The equations for calculating load equivalency factors are given in the Highway Research Board's report of the proceedings of the 1962 St. Louis Conference on the AASHO Road Test.[10]

The 1993 AASHTO Guide[1] states that, as a general rule, a given magnitude of rigid ESALs is approximately 50 percent higher than the equivalent magnitude of flexible ESALs. That is, for example, 15 million rigid ESALs are approximately equivalent to 10 million flexible ESALs. This is only a rule of thumb; in reality the ratio of rigid ESALs to flexible ESALs varies with total truck axle volume and design serviceability loss.

The 1993 AASHTO Guide[1] does not present any method for determining equivalent numbers of rigid and flexible pavement ESALs, nor for establishing equivalency between concrete slab thickness (D) and flexible pavement Structural Number (SN). The Guide suggests as a starting point that load equivalency factors for a 9-inch slab and and SN of 5 inches be selected to compute the design ESALs for any project, and that if the design obtained is appreciably different than the one initially assumed (more than an inch of concrete or asphalt), the design process should be repeated iteratively, until the load equivalency factors used yield thicknesses consistent with the thicknesses to which those load equivalency factors apply. In practice, this type of iterative design is rarely if ever done. A rule of thumb for approximate equivalency is $SN \times 2 \approx D$.

If either site-specific or general truck axle frequency distribution data are available, equivalent slab thicknesses and Structural Numbers can be computed for any design terminal serviceability and total truck axle volume, by the following procedure.

1. Use the total truck axle volume and truck axle frequency distribution information to determine the number of axles of each type (single, tandem, tridem) in each axle load group.

2. For the selected design terminal serviceability, compute the rigid and flexible load equivalency factors corresponding to the midrange of each axle load group considered.

3. Multiply the number of axles in each load group by the rigid and flexible load equivalency factors calculated for that load group, and sum the rigid and flexible ESALs calculated in each load group to determine the total rigid and flexible ESALs.

4. Solve for the concrete slab thickness and the flexible pavement Structural Number that both yield 1:1 ratios between the total ESALs computed in Step 3 and the ESALs computed from the basic AASHTO design equations[10] for an 18-kip single-axle. The rigid and flexible ESALs thus obtained are equivalent.

Distress Survey

Rehabilitation of a pavement is most likely to be successful – that is, provide satisfactory performance and cost-effectiveness – if it is selected on the basis of knowledge of the types of distresses occurring in the pavement and the causes for those distresses, and it effectively repairs those distresses. A good understanding of the types of distress which may occur in different types of pavements, and the causes for those distresses, is therefore essential to the success of pavement rehabilitation.

The different types of distresses which occur in asphalt, concrete, and asphalt-overlaid concrete road and highway pavements are briefly summarized in Tables 1, 2, and 3. More detailed descriptions of these distresses and their causes are provided in Appendix A of this Guide. Distresses typically seen in asphalt and concrete shoulders, which may require correction as part of the rehabilitation strategy for the mainline pavement, are also described in Appendix A.

Table 1. Asphalt pavement distress types and causes.

Distress	Causes	Comments
Fatigue cracking, also called alligator cracking	Fatigue damage in the asphalt concrete surface or stabilized base	Can progress to potholes, beginning first at locations where the underlying base and subgrade materials are weakest.
Block cracking and thermal cracking	An asphalt cement which is or has become too stiff for the climate. Asphalt concrete mixes subjected to low traffic volumes may not densify sufficiently and may become brittle, which leads to block cracking. An asphalt concrete mix may also be excessively brittle if it is mixed too long at the hot mix batch plant, mixed too hot, or stored too long.	More often seen in large paved areas, such as parking lots and airport aprons, than on roads and streets which carry channelized traffic.
Longitudinal cracking	Inadequate compaction at the edges of longitudinal paving lanes, reflection of underlying old pavement edges or cracks in a stabilized base, or application of heavy loads or high tire pressures in rutted wheelpaths.	Longitudinal cracking in rutted wheelpaths is more likely when heavy loads or high tire pressures are applied during cold weather to a rutted pavement with a weak subgrade.
Shoving and corrugation	Shear flow or slippage between layers, due to inadequacies of the asphalt concrete mix.	In an unstable mix, shoving develops first in areas where vehicles move more slowly. Additional horizontal friction forces produced by vehicles braking or accelerating can produce corrugations in an unstable mix.
Bleeding	Excess of asphalt cement and/or insufficient air voids in the asphalt concrete mix.	Bleeding occurs in hot weather. Asphalt cement expands and fills the voids in the asphalt concrete mix, and is then exuded at the pavement surface. This process is not reversible.
Slippage cracking	Poor bond between the surface layer and underlying layer.	Slippage cracking occurs in areas where vehicles brake and turn.

Table 1 (continued). Asphalt pavement distress types and causes.

Distress	Causes	Comments
Rutting	Inadequate asphalt concrete mix design for the applied tire pressures, or permanent deformation in the base, subbase, or subgrade.	
Ravelling and weathering	Loss of bond between the aggregate and binder. This may be due to insufficient asphalt cement content, poor adhesion of the asphalt cement to the aggregate, hardening of the asphalt cement, or segregation or inadequate compaction of the asphalt concrete hot mix during construction.	Ravelling is loss of aggregate particles, weathering is loss of asphalt binder. Both may pose a safety hazard.
Pumping	Excess moisture in the pavement structure, erodible base or subgrade materials, and high volumes of high-speed, heavy wheel loads.	
Bumps, heaves, and settlements	Foundation movement (frost heave, swelling soil) or localized consolidation, such as may occur at culverts and bridge approaches.	Detract from riding comfort; at high severity may pose a safety hazard.

Table 2. Concrete pavement distress types and causes.

Distress	Causes	Comments
Linear cracking (transverse, longitudinal, or diagonal)	Fatigue damage, often in combination with slab curling and/or warping; drying shrinkage; improper transverse or longitudinal joint construction; or foundation movement.	Low-severity shrinkage cracks in JRCP and CRCP are not considered structural distress; medium- and high-severity deteriorated shrinkage cracks are. All severities of linear cracking are considered structural distress in JPCP.
Corner breaks	Fatigue damage, often in combination with slab curling and/or warping and/or erosion of support at slab corners.	
D cracking	Freeze-thaw damage in coarse aggregates.	
Alkali-aggregate distress	Compressive stress building up in slab, due to swelling of gel produced from reaction of certain siliceous and carbonate aggregates with alkalies in cement.	Alkali-aggregate reaction includes alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR).
Map cracking and crazing	Alkali-aggregate reaction or overfinishing.	
Scaling	Overfinishing, inadequate air entrainment, or reinforcing steel too close to the surface.	
Joint seal damage	Inappropriate sealant type, improper sealant reservoir dimensions for the sealant type, improper joint sealant installation, and/or aging.	Loss of adhesion of sealant to joint walls, extrusion of sealant from joint, infiltration of incompressibles, oxidation of sealant, and cohesive failure (splitting) of the sealant are all considered joint seal damage.
Joint spalling, also called joint deterioration	Compressive stress buildup in the slab (due incompressibles or alkali-aggregate reaction); D cracking; misaligned or corroded dowels; poorly consolidated concrete in vicinity of joint; or damage caused by joint sawing, joint cleaning, cold milling, or grinding.	

Table 2 (continued). Concrete pavement distress types and causes.

Distress	Causes	Comments
Blowups	Compressive stress buildup in the slab (due to infiltration of incompressibles, or alkali-aggregate reaction).	A blowup may occur as a shattering of the concrete for several feet on both sides of the joint, or an upward buckling of the slabs.
Pumping	Excess moisture in the pavement structure, erodible base or subgrade materials, and high volumes of high-speed, heavy wheel loads.	
Faulting	Pumping of water and fines back and forth under slab corners, erosion of support under the leave corner, buildup of fines under the approach corner.	
Curling/warping roughness	Moisture gradients through the slab thickness, daily and seasonal cycling of temperature gradients through the slab thickness, and/or permanent deformation caused by a temperature gradient in the slab during initial hardening.	
Bumps, heaves, and settlements	Foundation movement (frost heave, swelling soil) or localized consolidation, such as may occur at culverts and bridge approaches.	Detract from riding comfort; at high severity may pose a safety hazard.
Polishing	Abrasion by tires.	Polished wheelpaths may pose a wet-weather safety hazard.
Popouts	Freezing in coarse aggregates near the concrete surface.	A cosmetic problem rarely warranting repair.

Table 3. Asphalt-overlaid concrete pavement distress types and causes.

Distress	Causes	Comments
Reflection cracking	Horizontal and differential vertical movements of joints and cracks in the concrete slab	
Fatigue cracking	Instability of the asphalt concrete mix coupled with loss of bond between the asphalt overlay and concrete slab.	Fatigue cracking in asphalt overlays of concrete looks like fatigue cracking in flexible pavements but occurs for different reasons.
Rutting	Lateral displacement due to shear stress in the asphalt concrete layer. Rutting may develop rapidly in an unstable mix.	Rutting in asphalt overlays of concrete looks like rutting in flexible pavements but occurs for different reasons.
Potholes	Reflection of localized failures in underlying concrete slab, or loss of bond between the asphalt overlay and concrete slab.	
D cracking	D cracking deterioration of underlying slab.	White fines pumping up through cracks in asphalt overlay suggest severe deterioration in slab.

A field survey is required to accurately determine the types, quantities, severities, and locations of distress present. Each of the distresses present may be indicative of rehabilitation needs and should be recorded by type, severity, and quantity in the distress survey.

The *LTPP Distress Identification Manual*[11] is widely used to guide field technicians in identifying distress types, rating distress severities, and measuring distress quantities on highway pavements. This manual, now in its third edition, is an update of the *Highway Pavement Distress Identification Manual,*[12] developed for FHWA and NCHRP. Another excellent distress identification guide for roadways is the one developed for U.S. Army Corps of Engineers´ PAVER system,[13] also summarized in *Pavement Management for Airports, Roads, and Parking Lots* by Shahin.[14] Distress types, severities, and measurement units are also described in Appendix K of the 1993 AASHTO Guide.[1] Similar distress identification manuals have been developed by several State DOTs.

The LTPP *Distress Identification Manual*[11] demonstrates how distresses may be mapped during a distress survey, as shown in Figures 2 and 3 for asphalt pavement, 4 and 5 for jointed concrete pavement, and 6 and 7 for continuously reinforced concrete pavement. Many State DOTs have their own standard forms for distress surveys. Suitable blank forms are also provided in the LTPP *Distress Identification Manual*.

For network-level management purposes, distress surveys are sometimes conducted over a sample of the full project length, e.g., 10 percent. For project-level purposes, however, sampling of a greater portion of the project length is necessary to accurately quantify the distress present. In some cases it may be advisable to sample the full project length, i.e., 100 percent.

Automated devices are also available for use in conducting distress surveys. These devices operate at highway speeds without disrupting traffic, and thus are particularly well suited to high-traffic-volume situations. Information on the capabilities of some automated distress survey devices is summarized in NCHRP Synthesis 203*, Current Practices in Determining Pavement Condition*.[15]

Nondestructive Deflection Testing

While some agencies may not be equipped for nondestructive deflection testing, such testing is always highly desirable, especially when the distress survey indicates that the pavement requires a structural improvement. A Falling Weight Deflectometer (FWD) or other device capable of applying loads comparable in magnitude to truck wheel loads is recommended for this purpose.

Nondestructive deflection testing devices are classified according to loading method: static (e.g., Benkelman Beam), vibratory (e.g., Dynaflect, Road Rater), or impulse, commonly called falling weight deflectometers or FWDs (e.g., Dynatest, KUAB). An example of a Dynatest FWD is shown in Figure 8, and an example of a KUAB FWD is shown in Figure 9.

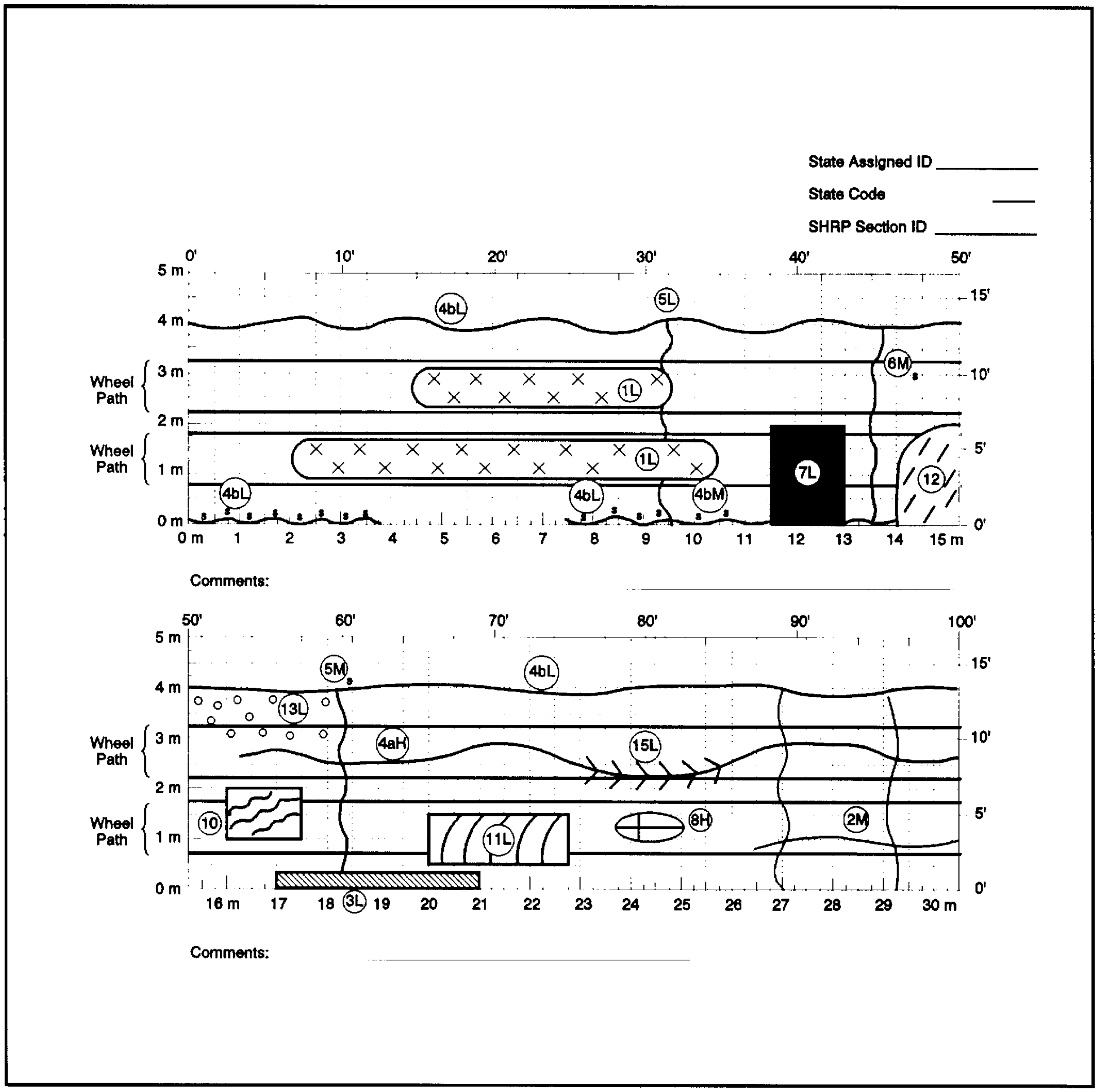

Figure 2. Example distress survey map for asphalt pavement.[11]

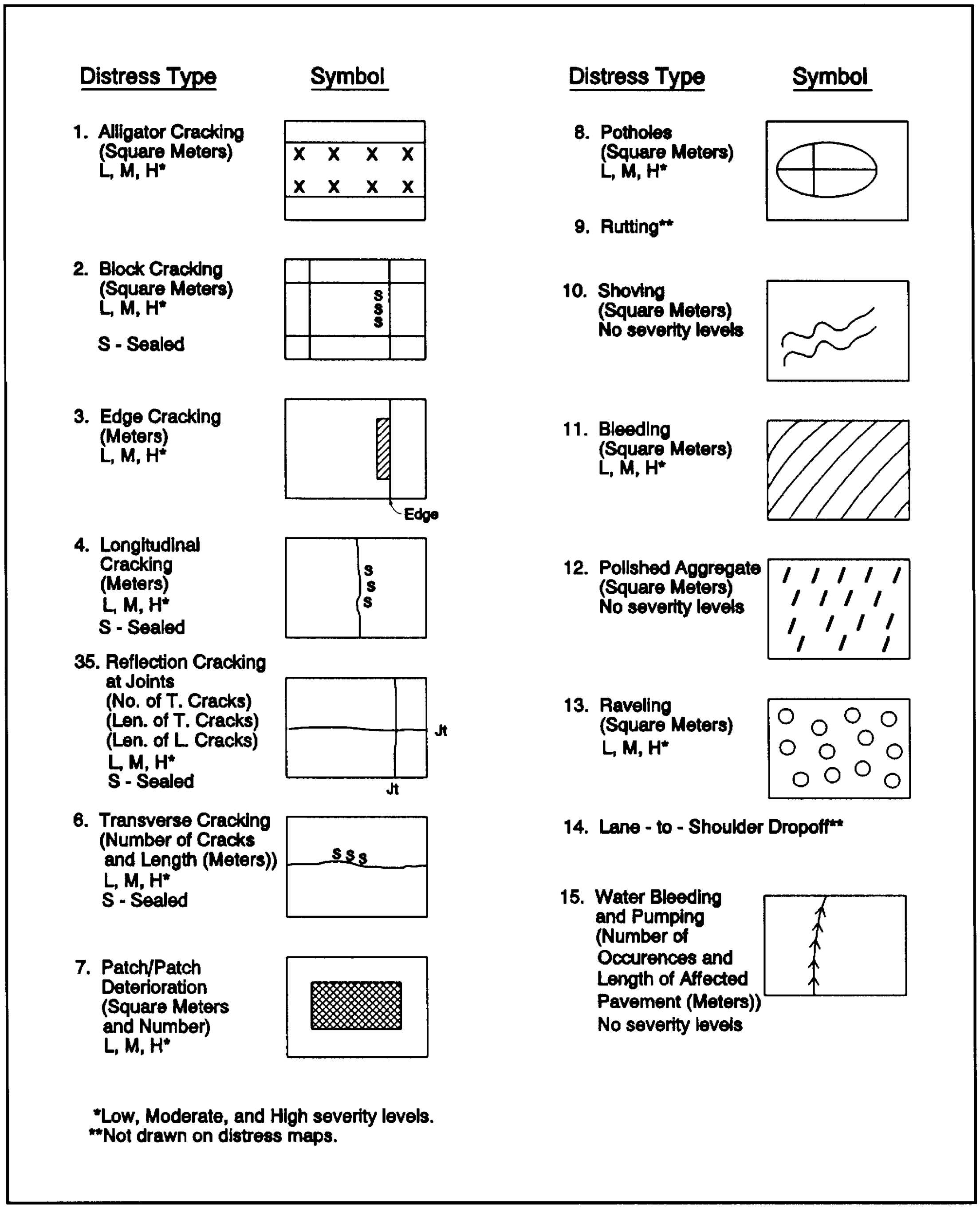

Figure 3. Symbols used in distress survey map for asphalt pavement.[11]

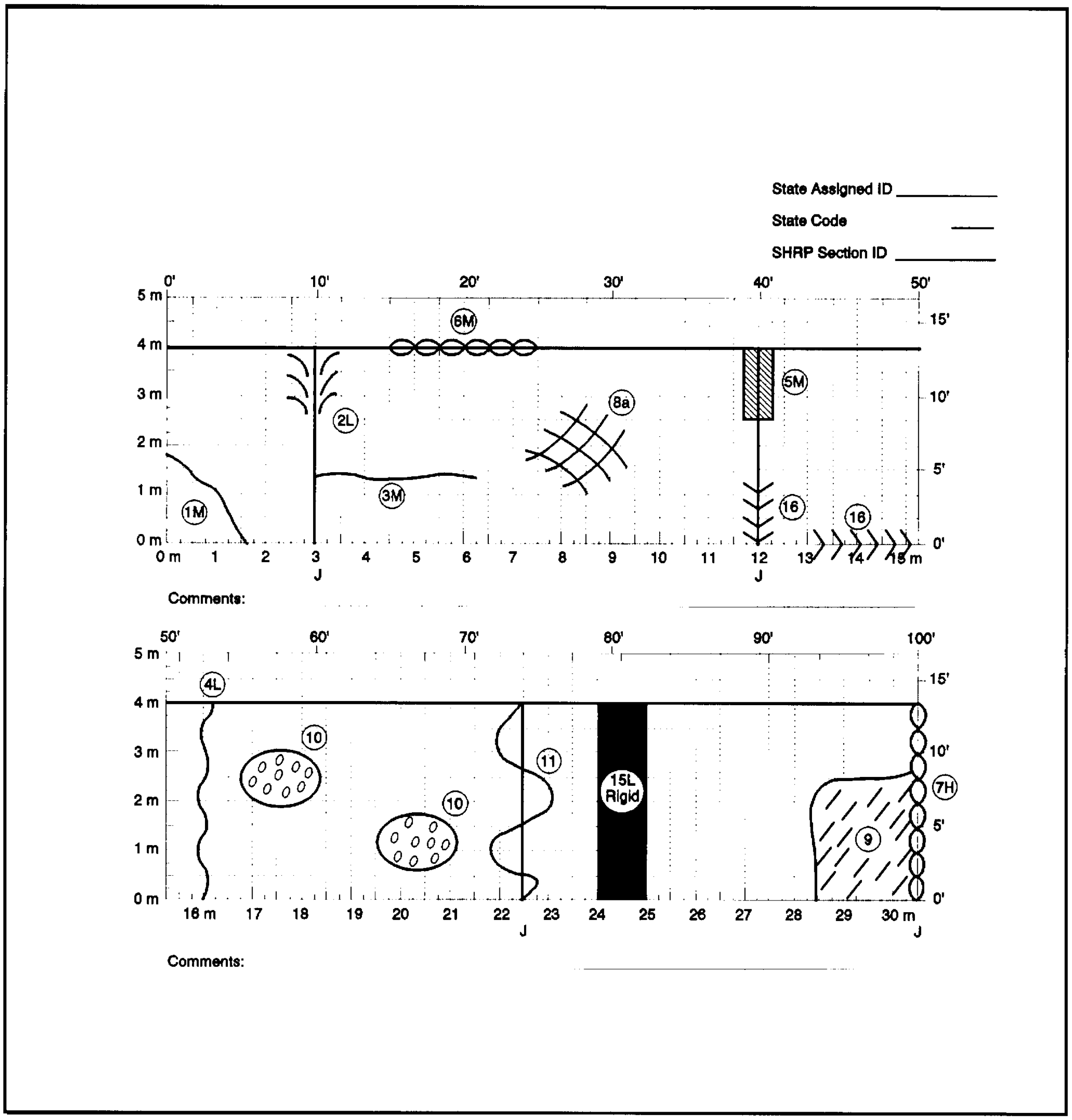

Figure 4. Example distress survey map for jointed concrete pavement.[11]

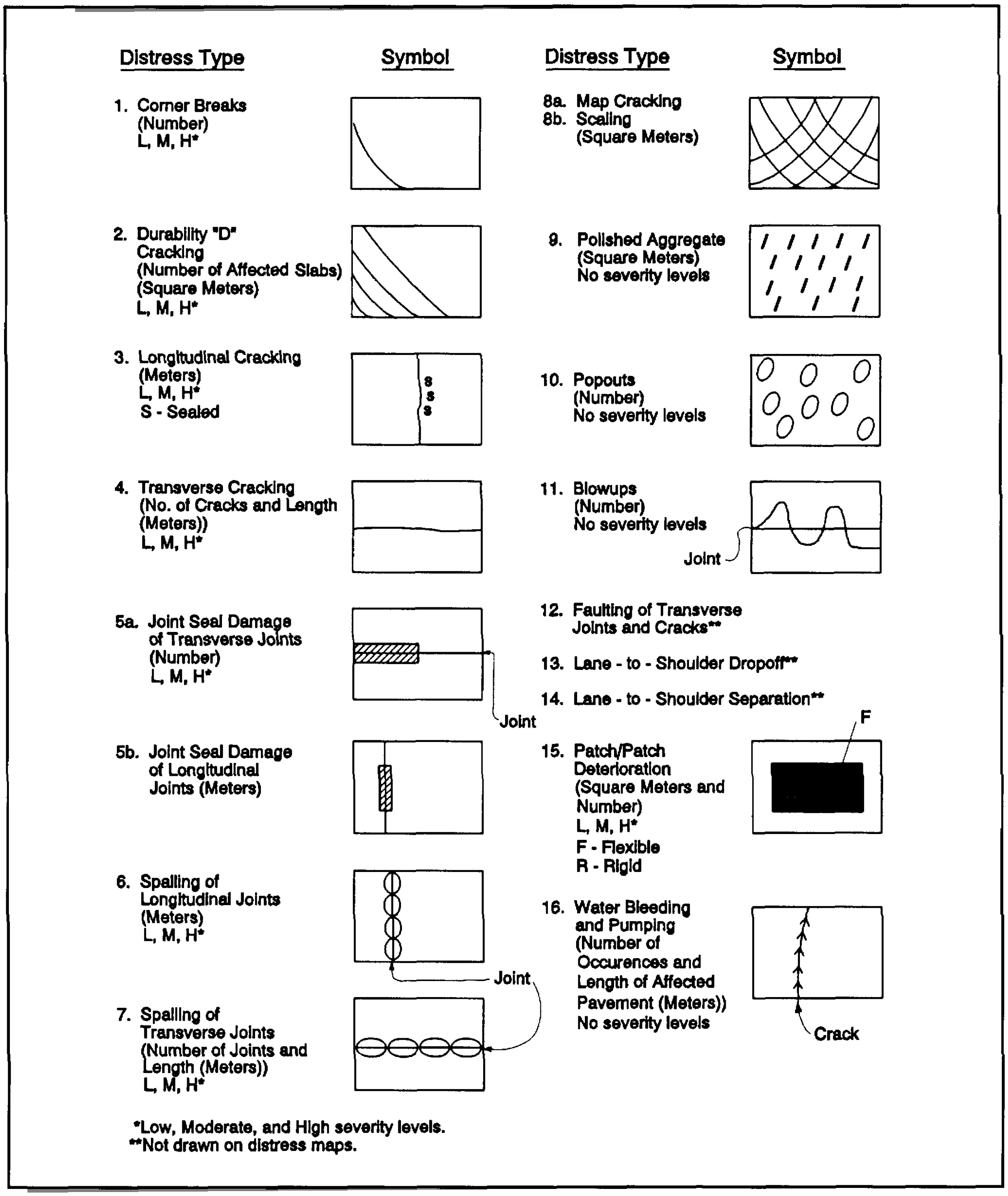

Figure 5. Symbols used in distress survey map for jointed concrete pavement.[11]

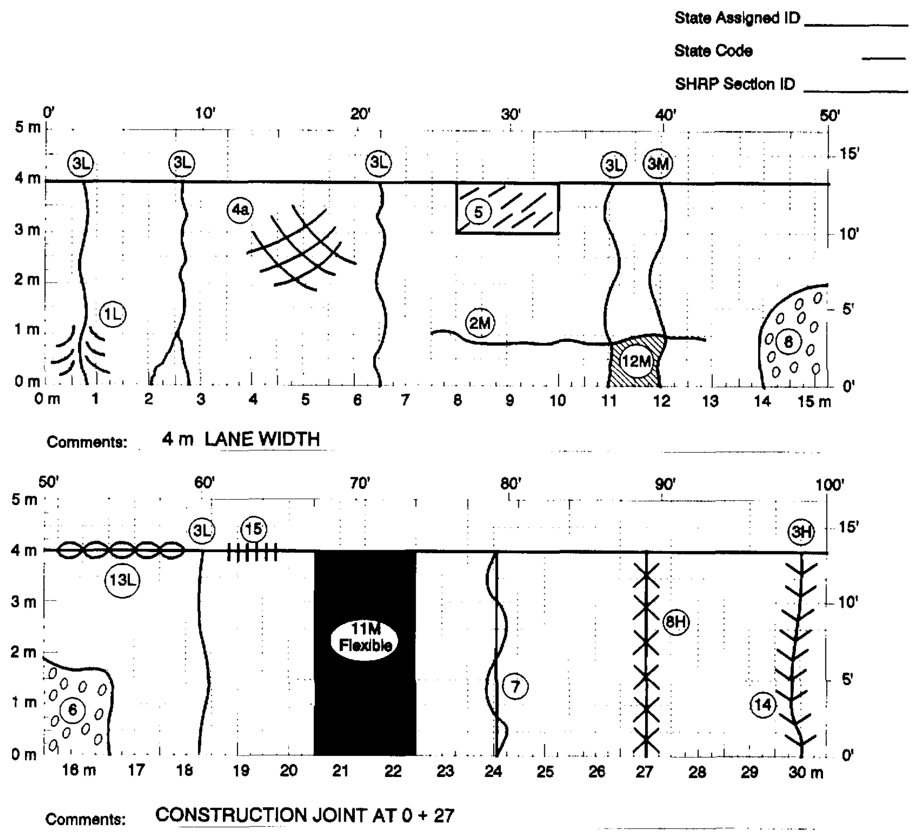

Figure 6. Example distress survey map for continuously reinforced concrete pavement.[11]

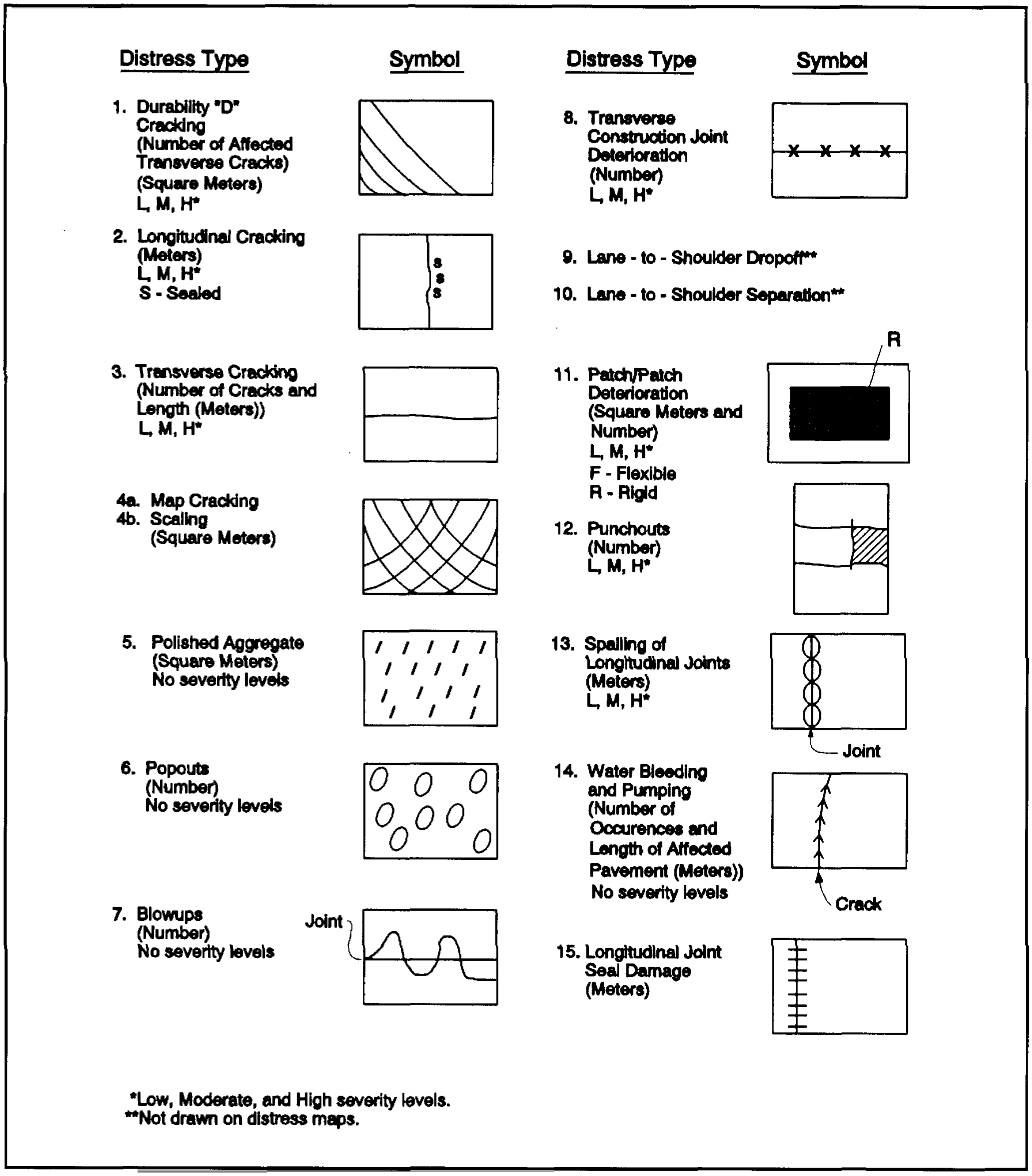

Figure 7. Symbols used in distress survey map for continuously reinforced concrete pavement.[11]

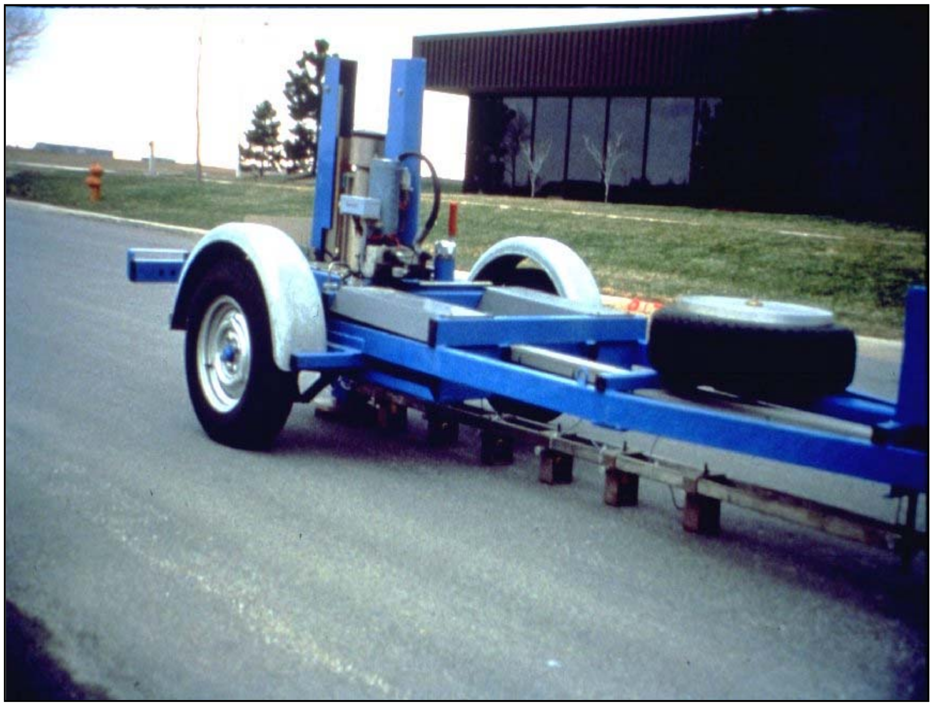

Figure 8. Dynatest FWD.

Figure 9. KUAB FWD.

The magnitude of the impact load applied to the pavement surface by a falling weight deflectometer depends on the number of masses dropped and the height from which they are dropped. The load magnitude is measured by a load cell at the center of the load plate. The deflections produced at the center of the load plate and at various distances from the load plate are measured by transducers that have a resolution of 1 μm and a precision of 2 percent plus or minus 2 μm. The data measured by the load cell and deflection transducers are recorded by a computer in the tow vehicle. The operator assigns a station number (e.g., 0+00) to the load plate position at the beginning of testing, and the positions of all deflection tests are subseqently recorded by coordination with the tow vehicle's distance measuring device. Closer views of the FWD load package, load plate, and sensors are shown in Figures 10 and 11.

Figure 10. Load package and sensor bar on falling weight deflectometer.

There are two categories of deflectometers, regular and heavyweight. Regular deflectometers are capable of applying loads in the range of 7 to 120 kN. They are suitable for testing on all roads and highways, as well as many airport pavements. In general, flexible pavements of any thickness, and rigid and composite pavements up to about 15 inches thick, can be tested with regular deflectometers. Testing on thicker rigid and composite pavements requires heavyweight deflectometers, which are capable of applying loads in the range of 30 to 240 kN.

Both lightweight and heavyweight deflectometers come equipped with two load plates, one 30 cm in diameter and the other 45 cm in diameter. The small load plate is used in highway pavement testing and most airport pavement testing. The large load plate is used in some airport pavement testing, to simulate loadings by aircraft with large gear assemblies. The large load plate is also sometimes used to conduct deflection testing directly on a subgrade or base layer.

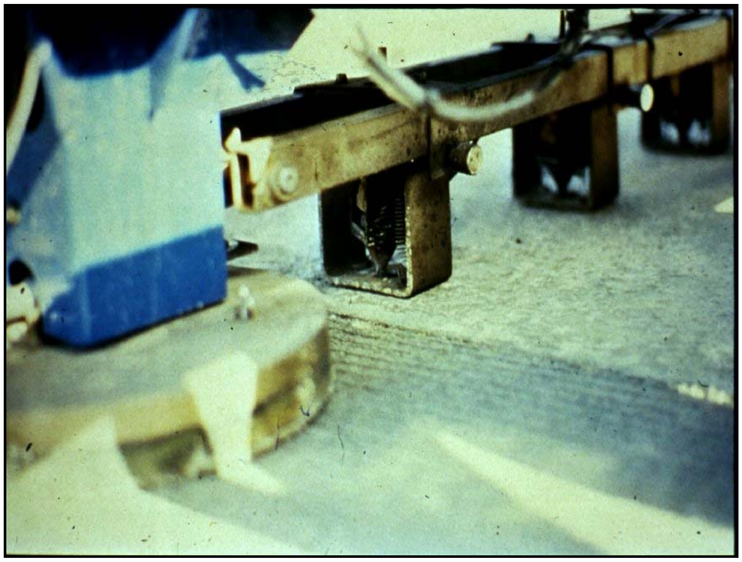

Figure 11. Falling weight deflectometer load plate and deflection sensors.

Overviews of the different devices available for nondestructive deflection testing are provided in *Pavement Management for Airports, Roads, and Parking Lots* by Shahin,[14] the FHWA report *Synthesis Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements*,[16] the FHWA report *Evaluation of Pavement Deflection Measurement Equipment,*[17] and the National Highway Institute's *Techniques for Pavement Rehabilitation* manual.[18]

General guidelines for deflection testing are given in ASTM D4695, *Standard Guide for General Pavement Deflection Measurements*. Guidelines for testing with falling weight deflectometer devices are given in ASTM D4694, *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device*. Guidelines for deflection testing are also given in the 1993 AASHTO Guide,[1] the *Techniques for Pavement Rehabilitation Manual*,[18] and by Shahin[14] and Hall.[19]

Testing on multilane highway pavements is usually done in the outer traffic lane only, because the outer traffic lane carries most of the truck traffic and thus typically exhibits more load-associated distress than the inner lane. In addition, closing the outer lane for deflection testing is considered safer than closing the inner lane, due to the perception that users are more accustomed to and better able to respond to closures in the outer lane. Deflection testing in the inner lane is usually only done when the inner lane is already closed for other reasons (e.g., repairs or bridge work).

For reasons of safety, nighttime testing on highway pavements is not recommended, although it may be necessary when (a) daytime traffic volumes are too high to permit access, or, (b) daytime slab temperature gradients are excessively high. Highway testing should also be avoided in conditions of fog, rain, or drizzle. As a rule of thumb, whenever vehicles have their headlights on, deflection testing is hazardous, and when the pavement is even slightly wet, it is even more hazardous.

Purposes of Deflection Testing

Deflection testing is conducted on asphalt pavements for the purposes of backcalculating the stiffnesses of the subgrade and pavement layers, assessing the remaining life of the pavement, and/or determining the overlay thickness required to satisfy a structural deficiency. Asphalt highway pavements should be tested in the outer wheel path of the outer traffic lane, which is just one to two feet from the lane edge, for the purpose of attempting to assess the extent of fatigue damage. The assumption of infinite horizontal layers is thus violated, but this is generally ignored.

Deflection testing on concrete pavements is conducted at slab interiors, to backcalculate the stiffnesses of the subgrade and pavement layers; at transverse and longitudinal joints and cracks, to measure deflection load transfer and differential deflection, and at slab corners, to detect voids under the slabs.

On concrete highway pavements, slab interiors are usually tested at the middle of the outer lane, for the purpose of backcalculating the dynamic modulus of subgrade reaction (k value) and concrete elastic modulus. A concrete slab of highway lane width (typically 12 ft) is narrower than that required to comply with the infinite horizontal layer assumption, so adjustments for the finite slab size are required when analyzing the deflection data. Interior deflections are not measured on concrete slabs with the goal of directly assessing the fatigue damage, so testing at the midwidth of the slab is no different than testing in the outer wheelpath, and may be preferable from the standpoint of keeping the load plate as far away as possible from the lane/shoulder edge. Testing in the outer wheel path may be more convenient, however, if interior tests and joint load transfer tests are to be combined in one pass down the traffic lane.

Deflection load transfer is measured for use in estimating the distribution of stress between adjacent slabs, which may be used in a mechanistic analysis of the fatigue life of the pavement. Deflection load transfer is also considered to be related to the development of faulting at joints and cracks. The deflection load transfer at transverse joints may also be used to select a load transfer coefficient (J factor) for use in the 1993 AASHTO method of overlay design. For all of these purposes, however, deflection load transfer measurements need to be adjusted for slab temperature in order to be meaningful.

One set of deflection measurements can be used to calculate both differential deflection (loaded side deflection minus unloaded side deflection) and deflection load transfer (ratio of loaded side deflection to unloaded side deflection). Differential deflection is more relevant than the deflection load transfer to the rate of deterioration of joints and cracks, and to the likelihood of reflection cracking in asphalt overlays.

Corner testing for void detection is the least commonly conducted of the three types of concrete pavement deflection testing. It may be warranted if the pavement already has some corner breaks, has very poor transverse joint load transfer, or manifests other signs of loss of support (e.g., pumping of fines or water at joints). However, to avoid errors in detecting voids, care must be taken to conduct void detection testing at times of the day or night when the slabs are flat (which may not necessarily coincide with when there is no temperature gradient through the slab thickness).

On asphalt-overlaid concrete pavements, deflections are measured at slab interiors to backcalculate layer and foundation stiffnesses. To measure deflection load transfer and differential deflection, deflections are measured at transverse and longitudinal joints and cracks. Deflection load transfer is conceivably useful in a mechanistic analysis of the asphalt-overlaid concrete pavement's remaining life, although this is not a common or straightforward analysis. The differential deflection is useful in identifying the joints and cracks deteriorating most rapidly. No accepted procedures have yet been established for void detection testing on asphalt-overlaid pavements.

Deflection Testing Interval

Typical testing intervals for highway pavements are between 100 and 500 ft (between about 50 and 10 points per mile, respectively). Deflections in asphalt pavements, and to some extent, in concrete pavements, tend to become more variable with time. Thus, a longer testing interval is appropriate for younger pavements, and a shorter interval is more appropriate for older pavements.

Measurement and Consideration of Temperature

Asphalt mix temperature measurements are required when testing asphalt and asphalt-overlaid pavements because the resilient modulus of asphalt concrete varies substantially with temperature. It is not uncommon for the AC mix temperature to vary by 30ºF or more during a typical day of deflection testing. This magnitude of temperature variation could easily correspond to a variation of 500,000 psi in asphalt concrete modulus. Failure to account for this variation will result in incorrect moduli being used for the asphalt layers.

The temperature at the middepth of the asphalt mix should be measured at least three times during each day of testing, to establish a curve of mix temperature versus time that may later be coordinated with the times recorded in the deflection data file. The air and surface temperatures are usually measured at the same time. Mix temperature is influenced by sunshine as well as air temperature. If parts of the pavement are shaded and others are not, temperatures should be measured in both shaded and unshaded areas, and it should be noted at each deflection location whether the location is shaded or unshaded. If it is not possible to obtain mix temperature measurements, the mix temperature may be estimated from air and surface temperatures, using procedures developed by Southgate,[20] Shell,[21] the Asphalt Institute,[27] Hoffman and Thompson, [22] or Lukanen et al.[23]

Backcalculation of asphalt pavement layer stiffnesses becomes difficult when the asphalt concrete modulus is greater than 2 million psi or less than 200,000 psi. Therefore, testing should be done when the temperature (measured in the asphalt concrete) is between 40^0F and 100^0F. When testing in sun and hot temperatures, the temperature of the asphalt concrete may be much higher than the air temperature. If mix temperature measurement is not possible, the pavement surface temperature (measured with an infrared gun) is better than the air temperature as an approximate indicator of whether the asphalt concrete temperature is too high for testing.

Temperature measurement is required when testing concrete pavements to monitor the temperature gradient in the concrete, and to relate the load transfer measurements to the temperature. The temperature gradient is monitored by measuring the temperature at three depths (for example, one quarter, one half, and three quarters of the slab thickness), at least three times during each day of testing, to establish a curve of temperature gradients versus time that may later be coordinated with the times recorded in the deflection data file.

Careful thought should be given to the allowable temperature range for testing concrete pavements for load transfer measurement purposes. Load transfer is highly dependent on temperature. Deflection load transfer (the ratio of unloaded side deflection to loaded side deflection, expressed as a percentage) follows an S-shaped curve, asymptotically approaching 100 percent at high temperatures, and a minimum percentage (greater than 0) at low temperatures. The full temperature-load transfer curve cannot be extrapolated from load transfer measured at only one temperature. This curve can be established, however, using measurements at a few selected reference points, at two significantly different temperatures, e.g., 20ºF or more apart.

Load transfer testing should be avoided when slab temperatures are so warm that the joints and cracks are closed. At what temperature this will occur depends on the joint/crack spacing. When testing in hot weather, measured load transfers should be checked in the field to see if they are very close to 100 percent. If they are, further load transfer testing should be postponed to some later time when the slabs are cooler and the joints have opened somewhat.

For concrete pavements, it is traditionally recommended to avoid testing for backcalculation purposes when a significant temperature gradient exists through the slab thickness. This is often taken to mean avoiding testing in the slab interiors during certain hours of the afternoon when the top of the slab is hotter than the bottom and the slab is curled downward, and avoiding testing at the edges and corners during certain hours of the night when the top of the slab is cooler than the bottom and the slab is curled upward. Whether or not a significant temperature gradient exists in the slab should be determined by measurement, as described earlier.

The real concern, however, is not merely whether the slab is curled, but whether the slab is curled out of contact with the underlying foundation. A slab resting on a soft foundation may be curled upward or downward and still be in full contact with the foundation at the location at which the deflections are measured. In many cases when the slab rests on a soil, gravel, or weakly stabilized subbase, the slab will not curl out of contact. The backcalculated slab and foundation moduli will be the same as if the deflections were measured while the slab was flat. However, when the slab rests on a high-strength stabilized base, the potential for curling out of contact is a concern. If the slab is curled out of contact with the foundation at the location where the deflections are measured (e.g., curled up and tested at the edge or corner, or curled down and tested at the midslab interior), then the backcalculated foundation modulus will be erroneously low.   Procedures have been developed for analyzing a series of deflections measured at different load levels, to determine when the slab is in contact with the foundation.[24]

It is not necessarily true that a slab is flat when it has no temperature gradient through its thickness. It is conceivable that a slab may be curled with a zero temperature gradient. This may occur if a temperature gradient existed in the slab during its initial set when it was constructed. Although multidepth temperature measurement is always recommended when testing concrete slabs, the more reliable way to ascertain whether or not a slab is in contact with the foundation at a given location is to conduct a load sweep, just as is done for void detection at corners. A series of loads of inccreasing magnitude are applied, and the relationship of load magnitude to deflection at the center of the load plate is examined. A straight-line relationship between more than two load-deflection points indicate that an elastic response has been achieved, signifying that at those load levels the slab is in contact with the foundation. Deflections measured at sufficiently high load levels to insure slab-foundation contact may indeed be used in backcalculation of the slab and foundation moduli.

The traditional method of testing for voids has been to use a load sweep at slab corners and assess the linearity and intercept of the load-deflection plot. This has been done on many projects without regard to whether or not the slabs were curled at the time of testing. Guidelines for evaluating corner deflections to distinguish curling from loss of support have been developed by Crovetti.[24,25]

Coordination of Deflection Testing with Visible Distresses

Asphalt pavements with alligator cracking in the wheelpaths may show significant variability in deflections and also in the degree of distress along the length of the project. A correlation can usually be observed between the severity of the alligator cracking and the magnitude of the maximum deflection. Assuming that one of the primary purposes of the deflection testing of an asphalt pavement is to assess its structural condition, it is useful to test at locations with various degrees of cracking. Even severely alligator-cracked areas can usually be tested.

It is more difficult to relate deflection magnitude to cracking in a concrete or asphalt-overlaid concrete pavement. One possible option is to measure deflections at the preselected interval and test in both cracked and uncracked areas (an area in the interior of the slab, away from joints or edges, without linear cracks or localized failures within the deflection basin). If deflection basins are measured this way in both cracked and uncracked areas, the concrete modulus backcalculated from the deflections should be considered an "effective" modulus, which represents not the true stress-strain behavior of intact concrete but rather the condition of the slab in its current state of cracking. An example of this approach to structural evaluation is found in the work done by Rollings,[26] in which a relationship was established between the "E ratio" (intact slab modulus versus cracked slab effective modulus) and the Structural Condition

Index determined from cracking data. However, measuring concrete or asphalt-overlaid concrete pavement deflections near cracks poses several practical difficulties. Different measurements will be obtained depending on where the FWD load plate is with respect to the crack.

The alternative is to backcalculate the concrete modulus only from deflection basins measured away from cracks. This modulus should not then be considered an indicator of the degree of structural damage in the slab. The backcalculation results and the distress survey results must then be considered together to form an overall assessment of the structural condition of the slab.

Target Load Levels

At least two and often three target load levels are used in deflection testing. One of the reasons for testing over a range of load magnitudes is to analyze whether or not the foundation exhibits a nonlinear response to load. Another reason is to be confident of obtaining at least one deflection basin of sufficient curvature for successful backcalculation. As a rule of thumb, a target load of sufficient magnitude to produce a mean maximum deflection of 6 mils is needed to obtain deflection basins of sufficient curvature to lend themselves to successful backcalculation. For highway pavements, at least one of the target load levels should be 9000 pounds, to facilitate an analysis of the pavement's structural capacity using the 1993 AASHTO Guide method.[1] Suggested target load levels for highway pavements are 6000, 9000, and 12000 pounds.

Number of Drops per Load Level

After the FWD is positioned at a station, a small amount of weight is dropped to insure that the load plate is properly seated on the pavement. If it is not (because, for example, a rock is under the plate), an error message from the computer will alert the FWD operator. This seating drop is not recorded with the load and deflection data.

After the seating drop, it is common practice when testing asphalt pavements to apply multiple load drops for each load level at each station testing. ASTM D4694 recommends that if significant permanent deformation under the loading plate occurs, the FWD should be moved to a different position and the applied force should be reduced "until the permanent deformation is of no significance to the first test at a test location."

For concrete and asphalt-overlaid concrete pavements, there is generally little or no significant change in deflections between load drops at the same load level. Two drops per load level are sufficient for these pavement types, the second one serving as a safeguard against a deflection sensor malfunctioning.

Sensor Configuration

Different sensor configurations yield different backcalculation results. Specifically, two configuration issues that significantly influence the magnitudes of the modulus values obtained from backcalculation are the outer radius to which the deflection basin is measured, and whether or not the maximum deflection (d_0 measured at the center of the load plate) is used in the backcalculation.

The main reason for measuring as far out as possible is to obtain one or more deflections far enough away from the load plate to estimate of the subgrade modulus independent of the effects of the overlying pavement layers. In selecting the distance for the farthest measurement point, consideration should be given to the fact that deflections decrease with distance but measurement error remains essentially constant. Thus the influence of measurement error on backcalculation becomes greater at greater distances from the load plate.

Most FWDs have at least six deflection transducers in addition to the transducer in the middle of the load plate. All seven deflection transducers should be used, and positioned to include at least one measurement beyond 36 inches. For highway pavements, a sensor configuration of 0, 8, 12, 18, 24, 36, and 60 inches is commonly used.

Load Transfer Measurement

When testing on concrete or asphalt-overlaid concrete pavement, one of the transducers in front of the load plate can be moved behind the load plate to measure load transfer on both the approach and leave sides of transverse joints and/or cracks. Similarly, to facilitate measurement of longitudinal load transfer (e.g., across the lane/shoulder joint on a concrete pavement with a tied concrete shoulder), another transducer can be mounted to one side of the load plate. When using the SHRP configuration (0, 8, 12, 18, 24, 36, and 60 inches), the transducers at 8 and/or 18 inches can be moved for these load transfer measurement purposes. This leaves in place the transducers at 0, 12, 24, and 36 inches for backcalculation, plus a distant sensor (60 inches) for an independent estimate of the subgrade modulus. Load transfer measurement across the longitudinal centerline joint or other longitudinal joints between highway traffic lanes is not recommended, for safety reasons.

At any given joint or crack in a concrete pavement or asphalt-overlaid concrete pavement, it is very possible that the load transfer measurements on the approach and leave sides will be unequal, because the crack in the slab rarely propagates completely vertically. However, on any given project, it may or may not be true that one side has consistently and significantly lower load transfer than the other side. If paired *t* tests show a significant consistent difference, the lower of the two should be used to compute the mean load transfer for use in slab stress analysis. It should also be kept in perspective that deflection load transfers are usually measured over a narrow temperature range during just a few days or hours out of the year and that all other load transfer levels for other temperatures are estimated.

Load transfer measurement is one of the more time-consuming aspects of deflection testing. It requires the operator to carefully position the load plate and sensors across the joint or crack, using either cameras mounted under the FWD or the help of an assistant. Measuring load transfer on both the approach and leave sides of transverse joints significantly increases the total testing time, and therefore is not recommended unless a specific objective of the project is to investigate differences in approach side and leave side load transfer. Otherwise, it is recommended to measure only the approach side load transfer (with the load plate behind the joint or crack), because it is then not necessary to move a deflection transducer to behind the load plate.

Materials Sampling and Testing

Any rehabilitation strategies involving overlay options will require information about the existing pavement materials and subgrade, for purposes of overlay thickness design. Depending on the design procedure used, the information required may include:

- Thicknesses of the pavement layers,
- Condition of the pavement layer materials,
- Elastic moduli of the pavement layers, and/or
- Elastic modulus or k value of the subgrade.

The stiffnesses of the pavement layers and subgrade may be determined from nondestructive deflection testing, as described previously. Layer thicknesses and stiffnesses may also be determined from laboratory testing of materials samples, or in some cases, from field tests. Materials sampling and testing is described in this section. Subgrade stiffness (elastic modulus or k value) may also be estimated from correlations with other soil properties, as described subsequently.

Layer Materials and Thicknesses

The material types and layer thicknesses should be determined from inventory records before deflection testing is conducted. Coring to check layer thicknesses may be done before deflection testing, but it is preferable to do coring after deflection testing, so that any unanticipated changes in the deflection magnitudes can be investigated during the coring. It is not usually feasible to conduct coring and deflection testing simultaneously, because the coring operation is slower and cannot keep pace with the deflection testing operation.

Layer thicknesses are important to the analysis of the deflection data. If a sound, full-thickness core cannot be obtained for a layer of asphalt or concrete (because the material is extensively deteriorated), the thickness can usually still be measured by probing in the core hole for the underside of the layer. Obviously, this condition should be recorded. If there are no plans to perform laboratory testing on the pavement layer materials and only the layer thicknesses are needed, a small-diameter (e.g., half-inch) drill bit may be used to determine asphalt, concrete, and stabilized base layer thicknesses.

It is also useful to note from observations of cores whether or not the layers are bonded together. However, layers that come out unbonded in the core may not have been unbonded in place; it is conceivable that the layers were separated by torsion during the coring operation. Examination of the interface may indicate whether the layer samples were separated during coring or had been unbonded for some time.

Asphalt Concrete Resilient Modulus Testing

Diametral resilient modulus testing may be conducted on cores from asphalt concrete and asphalt-treated base layers. Guidelines for this test are given in ASTM D4123, *Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures*. The test method involves repetitive loading along one diameter of core, and then along the perpendicular diameter. Resilient modulus testing usually involves tests at three temperatures, e.g., 41 ± 2, 77 ± 2, and 104 ± 2°F (5, 25, and 40 + 1°C), each at one or more loading frequencies, for example 0.33, 0.5, and 10 Hz. The recommended minimum core diameter is 4 inches for mixes with a maximum aggregate size up to 1 inch, and 6 inches for mixes with a maximum aggregate size up to 1.5 inches. The resilient modulus of the asphalt concrete is calculated as a function of the applied load, the thickness of the test sample, the measured recoverable horizontal deformation, and the Poisson's ratio (which may be calculated from the measured recoverable horizontal and vertical deformations, or assumed to be 0.35). The average resilient modulus is reported for each temperature, load duration, and load frequency used.

The elastic modulus of the asphalt concrete mix at any given temperature may be estimated using the following equation, which was developed by Witczak for use in the Asphalt Institute's MS-1 Design Manual.[27] This is a refinement of work originally done for the Asphalt Institute by Kallas and Shook.[28] It is considered highly reliable for dense-graded asphalt concrete mixes with gravel or crushed stone aggregates.[29]

$$\begin{aligned}\log E_{ac} = \; & 5.553833 + 0.028829 \, (P_{200} / F^{0.17033}) - 0.03476 \, V_v \\ & + 0.070377 \, \eta_{70°F,10^6} + 0.000005 \, t_p^{(1.3 + 0.49825 \log F)} \, P_{ac}^{0.5} \\ & - (0.00189 / F^{1.1}) \, t_p^{(1.3 + 0.49825 \log F)} \, P_{ac}^{0.5} \\ & + 0.931757 \, (1 / F^{0.02774}) \qquad \text{(Equation 1)}\end{aligned}$$

where:

E_{ac}	=	elastic modulus of the asphalt concrete, psi
P_{200}	=	percent of aggregate passing the No. 200 sieve
F	=	loading frequency, Hz
V_v	=	air voids, percent
$\eta_{70°F,10^6}$	=	absolute viscosity at 70°F, 10^6 poise
P_{ac}	=	asphalt content, percent by weight of mix
t_p	=	asphalt concrete mix temperature, °F

This equation can be reduced to a relationship between the asphalt concrete elastic modulus and the mix temperature for a particular loading frequency by measuring or assuming values for the asphalt cement and mix parameters in the equation. It should be noted, however, that the equation applies to new mixes. Asphalt concrete materials which have been in service for some time may have either a higher modulus (due to hardening of the asphalt) or a lower modulus (due to deterioration of the mix, from stripping or other causes) at any given temperature. Therefore, it is recommended that the results of diametral resilient modulus testing on cores at two or more temperatures be used to calibrate the above equation for the particular mix being evaluated.

It should also be remembered that the elastic modulus that an asphalt concrete mix exhibits in the field, i.e., under traffic loading or during nondestructive deflection testing, is typically about 2 to 2.5 times higher, at any given temperature, than the elastic modulus that will be measured for this same mix in the laboratory.[19] For example, a mix for which an elastic modulus of 400,000 psi is measured in the laboratory at 70°F may be expected to exhibit an elastic modulus of between about 800,000 and 1 million psi in the field at the same temperature. The reason for this is the difference between the load frequency in the laboratory test and the load frequency of nondestructive deflection testing, which simulates high-speed traffic loading. Laboratory diametral resilient modulus testing is typically conducted at frequency of about 1 to 2 Hz. The load duration of the falling weight deflectometer is about 25 to 30 milliseconds,[30] which corresponds to a loading frequency of between 15 and 20 Hz.

For a given set of asphalt concrete mix parameters, laboratory testing frequency, and deflectometer impulse load duration, Equation 1 may be used to calculate the ratio of field modulus to laboratory modulus at any given temperature. Whether the laboratory modulus or field modulus is used in analysis depends on the input required for the analysis model being used. For example, the laboratory modulus is the correct input to asphalt concrete fatigue models developed from laboratory testing, while the field modulus is the correct input to fatigue models developed from full-scale field testing.

Asphalt Concrete Indirect Tension Testing

Indirect tension testing uses the same test setup as diametral resilient modulus testing but involves applying a single load, at a constant rate of deformation, to failure of the sample. Guidelines for this test are given in ASTM D 4123*, Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures*. The indirect tensile strength is calculated as a function of the applied load, the length of the sample, and the diameter of the sample. Indirect tensile strength testing is faster than diametral resilient modulus testing, but may be less useful: the two have not been demonstrated to correlate well.

Marshall Stability and Flow Testing

Marshall stability and flow testing may be conducted on asphalt concrete cores to determine the stability and flow of the mix. Guidelines for this test are given in ASTM D5581, *Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus (6 inch-Diameter Specimen)*. The stability is the maximum load resistance in pounds that a specimen exhibits at 60°C. The flow is the strain (in units of 0.25 mm) measured during loading. The mix density, air voids, and maximum theoretical specific gravity must also be determined to relate the results of the stability-flow test to potential or observed problems such as excessive rutting.

Asphalt Extraction Testing

Extraction testing may be conducted on asphalt concrete and asphalt-treated base layer cores to separate the asphalt cement from the aggregate and subsequently determine the asphalt cement content and the aggregate gradation. The stiffness of the recovered asphalt cement may be determined from penetration or viscosity testing, as described in ASTM D2171, *Standard Test Method for Viscosity of Asphalts by Vacuum Capillary Viscometer*. This testing is especially important if consideration is being given to recycling some or all of the existing asphalt materials.

Concrete Indirect Tenstion Testing

Indirect tensile testing may also be conducted on cores from concrete layers. A widely used correlation between concrete indirect tensile strength (σ_t) and third-point modulus of rupture (S'_c) is the following, developed by Hammitt (all units are psi):[31]

$$S'_c = 1.02\,\sigma_t + 210 \qquad \text{(Equation 2)}$$

Soil Sampling

Samples of unbound base materials and subgrade materials may be obtained in the field for laboratory testing. Fine-grained soil samples may be obtained by split-spoon sampling, as described in ASTM D1586, *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils*. Granular material samples may be obtained by augering or trenching.

Resilient Modulus Testing of Soils

Resilient modulus testing may be conducted on fine-grained and coarse-grained soil samples. Guidelines for laboratory resilient modulus testing are given in AASHTO T294-92. A compacted soil sample is placed in a triaxial test apparatus, subjected to an all-around confining pressure, and further subjected to repeated axial load, while the resulting vertical deformation is measured. The resilient modulus of the soil is calculated as the ratio of the deviator stress (the total vertical stress minus the all-around confining pressure) to the resilient strain (that portion of the total strain which is recovered when the load is removed). A diagram of the resilient modulus test apparatus is shown in Figure 12. The concept of the resilient modulus is illustrated in Figure 13.

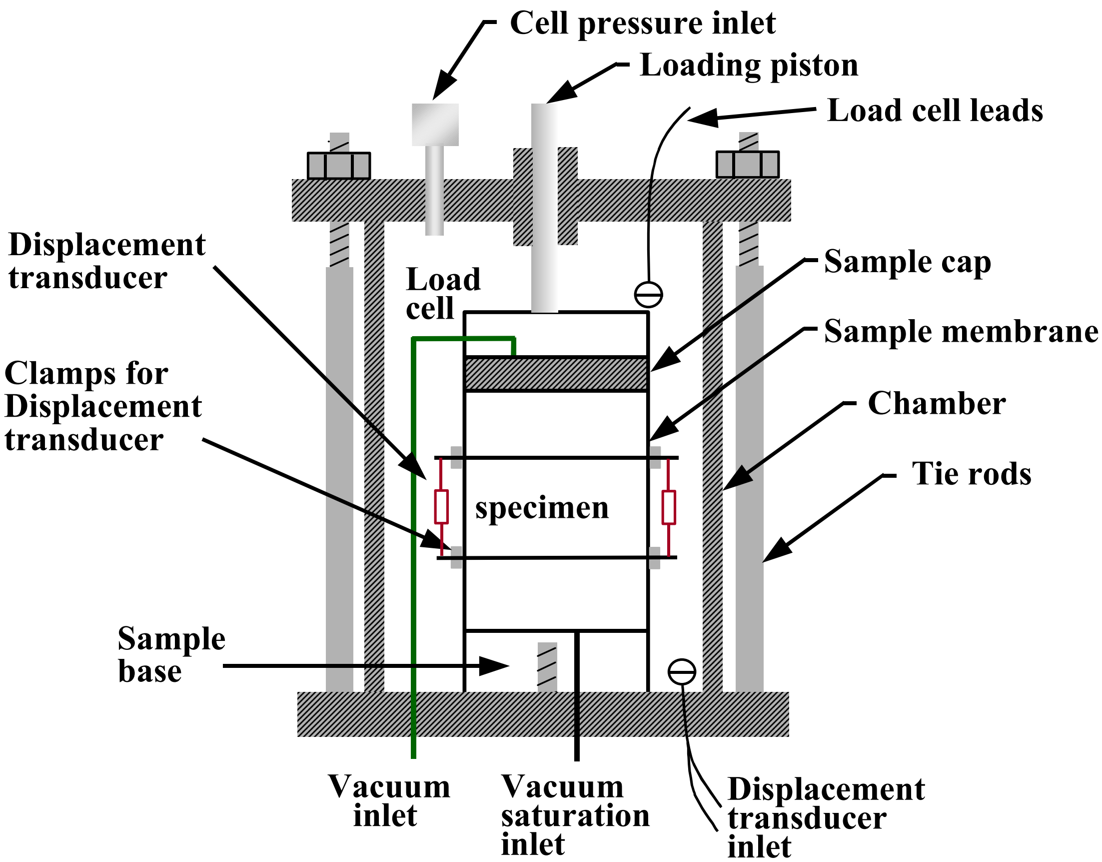

Figure 12. Resilient modulus testing apparatus for soils.[18]

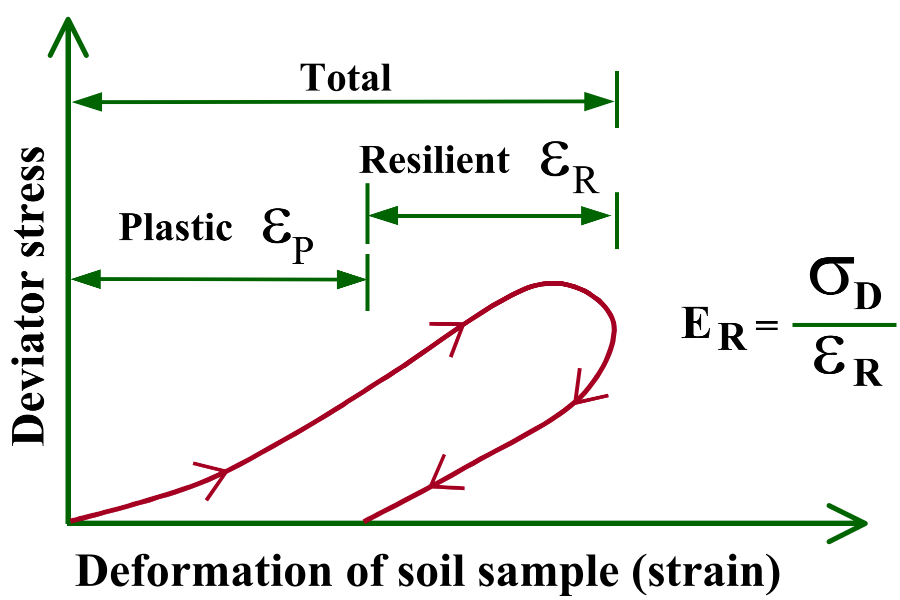

Figure 13. Resilient modulus concept.[18]

Fine-grained soils tend to exhibit stress-softening behavior, that is, the resilient modulus decreases with increasing deviator stress. Coarse-grained soils, on the other hand, tend to exhibit stress-hardening behavior: higher resilient modulus at higher deviator stress levels. The resilient modulus that a material exhibits in the laboratory may be considerably lower than the resilient modulus that the same material exhibits in the field, due to differences in the magnitudes of deviator stress, all-around confining pressure, and loading rate. Field resilient modulus values for fine-grained soils, obtained by backcalculation from falling weight deflectometer deflections, have been reported in a number of studies to exceed laboratory resilient modulus values by factors between about 3 and 5.[1] Less information is available about the relationship of field-to-lab resilient modulus for coarse-grained soils, but in general, field modulus values are expected to be higher than laboratory modulus values for these materials as well.

Whether the laboratory modulus or field modulus of the subgrade soil is used in analysis depends on the input required for the analysis model being used. For example, the original AASHO Road Test model for fexible pavement performance was calibrated to the laboratory resilient modulus of the soil at the AASHO Road Test site. Therefore, when using the 1993 AASHTO overlay procedure to determine the required asphalt overlay thickness for an in-service asphalt pavement, the appropriate input for the subgrade soil is the laboratory resilient modulus.

California Bearing Ratio Testing

The California Bearing Ratio (CBR) test is a simple laboratory test that measures the resistance of a soil sample to the penetration of a piston at a constant rate. Guidelines for CBR testing are given in AASHTO T 193. The CBR of the soil is the ratio, expressed as a percentage, of the load corresponding to a given penetration, to the load corresponding to the same penetration for a standard well-graded crushed stone.

Stabilometer Testing

The stabilometer test is a simple laboratory test that measures horizontal stress in a soil sample as a result of a constant vertical pressure. The resistance value (R value) is calculated as a function of the applied vertical pressure, the transmitted horizontal pressure, and the displacement of stabilometer fluid necessary to increase the horizontal pressure from 5 psi to 100 psi.

Soil Testing in the Field

The CBR test may also be conducted in the field, as described in AASHTO T 193. An efficient and inexpensive way to estimate the in-place CBR is with a Dynamic Cone Penetrometer (DCP). This device is a graduated rod with a metal cone on one end and a mass which is repeatedly lifted and dropped to drive the cone into the soil. The DCP's penetration rate (mm/blow) correlates well to CBR for fine-grained soils (CBR up to about 15 percent). A diagram of the DCP is shown in Figure 14.

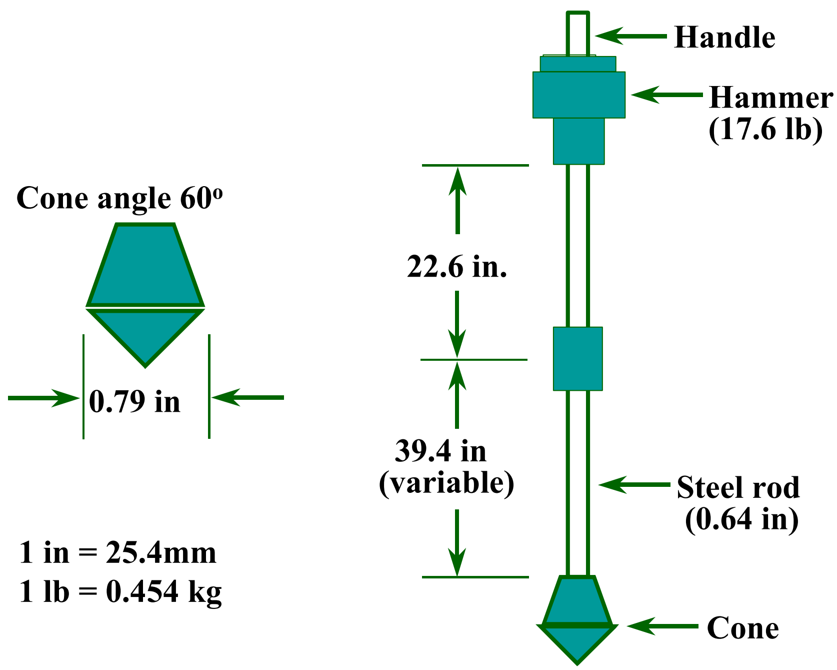

Figure 14. Diagram of Dynamic Cone Penetrometer (DCP).[18]

Plate load testing of subgrade soils is not commonly done because it is slow, labor-intensive, and in the case of existing pavements, requires removing segments of the surface and base layers. It is nonetheless the direct method for determining the static modulus of subgrade reaction (k value) which is a required input to concrete pavement overlay design procedures. A diagram and a photo of the plate load testing apparatus are shown in Figures 15 and 16 respectively.

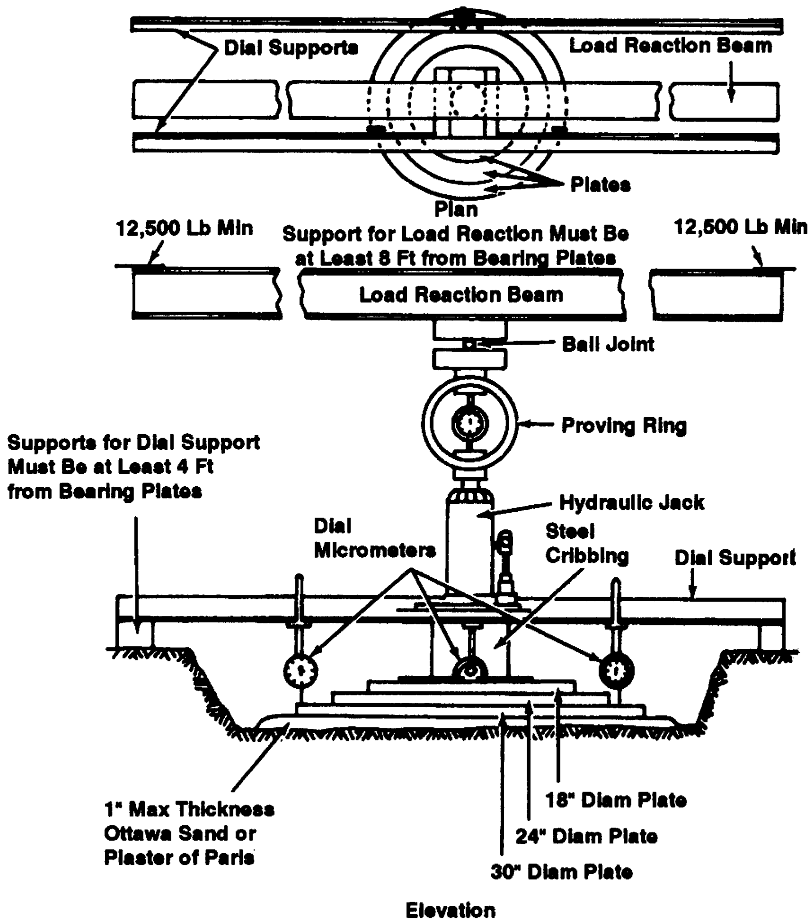

Figure 15. Diagram of plate load testing apparatus.[14]

Guidelines for repetitive static plate load testing are given in ASTM D1195, *Standard Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements,* and in AASHTO T221. In the repetitive test, the static elastic k value is calculated as the ratio of the applied pressure to the elastic deformation (the recoverable portion of the total deformation measured). Guidelines for nonrepetitive static plate load testing are given in ASTM D1196, *Standard Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements*, and in AASHTO T222. In the nonrepetitive test, the pressure-deformation ratio at a deformation of 0.05 inch is considered to represent the static elastic k value. A 30-inch-diameter plate should be used to determine the static elastic k value from either repetitive or nonrepetitive plate load testing. Smaller-diameter plates will yield higher k values that are inconsistent with the subgrade response to full-size slab loading.[32]

Figure 16. Photo of plate load testing apparatus.

The dynamic k value obtained by backcalculation from deflections measured on concrete slabs, using a falling weight deflectometer, is about 2 times greater than the static elastic k value that would be obtained from plate load testing of the same soil.[33] This is due to the difference in the soil's response to dynamic loading and static loading.

Correlations have been developed to estimate soil k values as a function of CBR, density, and soil class.[34,35,36,37,38] Several of these correlations are summarized in Table 4. Additional correlations between soil properties (gradation, density, moisture content), soil classification, CBR, DCP penetration rate, and resilient modulus, are given in the Illinois Department of Transportation's guidelines on subgrade inputs and subgrade stability requirements for local road pavement design.[39] More information on soil classification and soil properties is available from the Portland Cement Association[36] and the Asphalt Institute.[40]

Table 4. Correlation of k value to soil type, density, and CBR.[41]

AASHTO Class	Soil Description	Unified Class	Dry Density (lb/ft^3)	CBR (percent)	Static k value (psi/inch)
Coarse-grained soils:					
A-1-a, well graded	Gravel	GW, GP	125 – 140	60 – 80	300 – 450
A-1-a, poorly graded			120 – 130	35 – 60	300 – 400
A-1-b	Coarse sand	SW	110 – 130	20 – 40	200 – 400
A-3	Fine sand	SP	105 – 120	15 – 25	150 – 300
A-2 soils (granular materials with high fines):					
A-2-4, gravelly	Silty gravel	GM	130 – 145	40 – 80	300 – 500
A-2-5, gravelly	Silty sandy gravel				
A-2-4, sandy	Silty sand	SM	120 – 135	20 – 40	300 – 400
A-2-5, sandy	Silty gravelly sand				
A-2-6, gravelly	Clayey gravel	GC	120 – 140	20 – 40	200 – 450
A-2-7, gravelly	Clayey sandy gravel				
A-2-6, sandy	Clayey sand	SC	105 – 130	10 – 20	150 – 350
A-2-7, sandy	Clayey gravelly sand				
Fine-grained soils:					
A-4	silt	ML, OL	90 – 105	4 – 8	25 – 165 *
	Silt/sand/gravel mix		100 – 125	5 – 15	40 – 220 *
A-5	Poorly graded silt	MH	80 – 100	4 – 8	25 – 190 *
A-6	Plastic clay	CL	100 – 125	5 – 15	25 – 225 *
A-7-5	Moderately plastic elastic clay	CL, OL	90 – 125	4 – 15	25 – 215
A-7-6	Highly plastic elastic clay	CH, OH	80 – 110	3 – 5	40 – 220 *

* The k value of fine-grained soil is highly dependent on degree of saturation. Adjustment to k value is required for embankments less than 10 ft thick over a softer subgrade, and/or for bedrock at a depth within 10 ft.

Profile and Roughness Measurement

Roughness may be characterized by indices which are based on either the measured profile of the measured surface, or the output from a roughness meter installed in a vehicle. At the project level, roughness measurements can be useful in locating areas of excessive roughness, deciding whether or not a nonoverlay rehabilitation strategy should include some treatment for reducing roughness (such as an overlay or diamond grinding), and assessing the effectiveness of such treatments. In general, however, roughness measurement plays a larger role in network-level pavement management (i.e., identifying projects in need of maintenance or rehabilitation) than in project-level evaluation.

The measured profile may also be used to simultaneously produce, by simulation, the outputs of other roughness devices measuring devices as if those devices had been used to measure the surface. Simulation of vehicle responses from profile measurements is described in ASTM E1170, *Standard Practices for Simulating Vehicular Response to Longitudinal Profiles of Traveled Surfaces*. Devices that can be simulated include the BPR Roughometer, the CHLOE Profilometer, the Mays Ride Meter, the PCA Road Meter, and various straightedge devices.

Pavement profile measurement is described in ASTM E950, *Standard Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer-Established Inertial Profiling Reference*. Profile measurement is most efficiently done using a high-speed non-contact profilometer such as the K. J. Law profilometer or the South Dakota profiler. Profile measurement may also be done using survey rod and level equipment or a Dipstick device. Descriptions of profile and response-type roughness measurement devices are given by Shahin.[14]

The International Roughness Index (IRI) is a roughness parameter which is obtained from a mathematical model applied to a measured profile. The model simulates a quarter-car (one wheel) system travelling at 80 km/hr. The IRI is computed as the cumulative movement of the suspension of the quarter-car system divided by the travelled distance. Detailed guidelines on accurate measurement of longitudinal pavement profile for the purpose of calculating International Roughness Index (IRI) are given in NCHRP Report 434, *Guidelines for Longitudinal Pavement Profile Measurement*.[42]

Present serviceability index (PSI) may be estimated from International Roughness Index using the following equations,[43] derived from AASHO Road Test data:[44]

Asphalt pavement:

$$PSI = 5 - 0.2937\,x^4 + 1.1771\,x^3 - 1.4045\,x^2 - 1.5803\,x \qquad \text{(Equation 3)}$$

Concrete pavement:

$$PSI = 5 + 0.6046\,x^3 - 2.2217\,x^2 - 0.0434\,x \qquad \text{(Equation 4)}$$

Where in both cases:

x	=	log (1 +SV)	(Equation 5)
SV	=	2.2704 IRI^2	(Equation 6)
PSI	=	present serviceability index	
SV	=	slope variance (10^6 x population of variance of slopes at 1-ft intervals)	
IRI	=	International Roughness Index, m/km	

Models similar to those given above for estimation of present serviceabilty index have been developed by Dujisin and Arroyo,[45] and models for estimation of present serviceability rating (PSR) have been developed by Paterson,[46] Al-Omari and Darter,[47] and Gulen et al.[48] Several correlations between serviceability and IRI are illustrated in Figure 17.

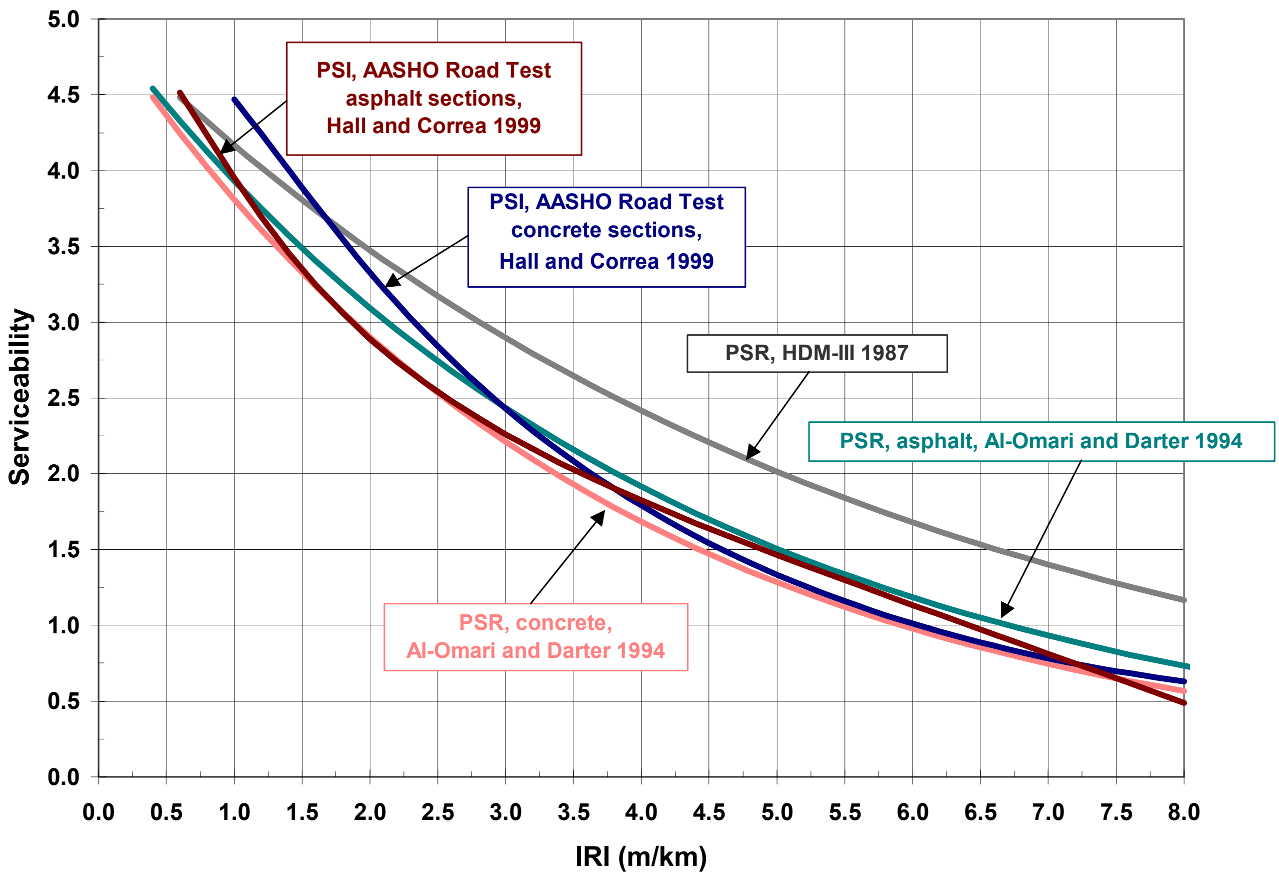

Figure 17. Models for estimation of serviceability (PSR or PSI) from IRI.[43]

Friction Measurement

Friction testing, like roughness testing, is less a project-level evaluation activity than a network-level pavement management activity. At the project level, friction measurements can be useful in deciding whether or not a nonoverlay rehabilitation strategy should include some treatment improving surface friction (such as an overlay, diamond grinding, or grooving), and assessing the effectiveness of such treatments.

Friction testing may be done using testing wheels, in locked-wheel mode, slip mode, or yaw mode, or using smaller laboratory devices. Locked-wheel mode testing uses a locked-wheel trailer towed behind a truck which sprays water on the pavement, as illustrated in Figure 18.

Figure 18. Locked-wheel friction testing.

The friction parameter obtained from locked-wheel testing is the Skid Number, or 100 times the friction coefficient measured. This test method is described in ASTM E274, *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. Slip mode testing involves measuring the change in angular wheel speed during braking of a free-rolling wheel. Yaw-mode testing involves turning the testing wheel (without braking) to some angle away from the direction of motion, and measuring the sideways friction factor.

Among the smaller devices available for measuring surface friction, the most commonly used is the British Portable Tester, the use of which is described in ASTM E303, *Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester*. Pavement surface texture can be measured by methods such as the sand patch method, in which a known volume of sand is spread over the surface and the area covered is measured. Laser devices such as the TRRL texture meter may also be used to measure surface texture. Descriptions of these and other friction and texture measurement methods are given by Shahin.[14] Additional information on pavement friction is given in NCHRP Synthesis 291, *Evaluation of Pavement Friction Characteristics.*[49]

Drainage Inspection

The most obvious signs of inadequate subsurface drainage will be notable during the distress survey: pumping of water and/or fines at transverse and/or longitudinal joints, blowholes along the lane/shoulder joint, and localized settlement of an asphalt concrete shoulder near blowholes. D-cracking in a concrete pavement may also indicate a drainage deficiency. A third major moisture-related problem is stripping in asphalt and asphalt-overlaid concrete pavements, which may be investigated by visual examination of cores after splitting.

The following additional indications of inadequate drainage should also be noted during the field survey:

- Standing water in the ditches,
- Cattails or other water-loving vegetation in the ditches,
- Inadequate height of subdrain outlets or daylighted base above the ditchline,
- Clogging or obstruction of subdrain outlets, or
- Clogging of daylighted base by soil and/or vegetation.

If visual observations suggest a significant drainage deficiency may exist, more intensive inspection may be conducted. The effectiveness of both longitudinal edgedrains and daylighted bases may be evaluated by using a truck to dump water on the pavement and observing the outflow, or by observing the outflow during or immediately after a rainfall. Localized clogging, obstruction, and crushing of longitudinal edgedrain pipes can be investigated using video inspection equipment.

Other Nondestructive Testing

Ground-Penetrating Radar

Ground-penetrating radar (GPR) is used to estimate pavement layer thicknesses, joint deterioration, moisture contents in base layers, and stripping in asphalt concrete layers. Ground-penetrating radar has been tested on asphalt, concrete, and asphalt-overlaid concrete pavements, as well as bridge decks.

Short-pulse, ground-penetrating radar works on the principle of wave propagation and reflection and transmission of electromagnetic waves. A brief pulse of electromagnetic energy is directed into the pavement. Dielectric discontinuities in the pavement (for example, changes in material type, moisture content, or density) cause part of the incident wave to be reflected and part to be transmitted into the next layer. This reflected energy is recorded by devices at the surface, and analyzed to determine pavement properties (layer thicknesses, voids, moisture contents, etc.). Vehicles equipped with ground-penetrating radar equipment (radar systems, transducers, antennae, and on-board recording devices such as magnetic tapes, oscilloscopes, and computer hardware and software) operate at speeds from 3 to 70 mph, which may require a moving lane closure. The data collection speed is a function of the antenna type used (air launched or ground coupled) and the frequency at which data are required. For project-level surveys, where one trace is required for every 6 ft or so of pavement, the data can be collected at highway speeds, but on bridge deck surveys with ground-coupled equipmenet, a lane closure may be required. Ground-penetrating radar testing is described in ASTM D4748, *Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar*.

The major advantages of ground-penetrating radar testing are its speed and accuracy. It continues to be the only technology which can provide meaningful subsurface information at close to highway speed. Its disadvantages include the complexity of the radar output and the lack of good software to convert the signals into information meaningful to pavement engineers. Current data analysis methods are labor intensive and require considerable expertise for interpretation of the raw data. Some coring is required with ground-penetrating radar, for calibration purposes.[50]

Infrared Thermography

Infrared thermography is used to locate reinforcing steel and detect concrete delaminations in reinforced concrete pavements. Infrared thermography has also been used to detect debonding at asphalt/concrete interfaces, and to measure temperature differentials in newly placed asphalt overlays. Developed initially for application to bridge deck inspection, infrared thermography has been used to survey pavements as well.

Infrared thermography is the process of detecting temperature differences associated with defective areas within a pavement. Various types of infrared scanners have been used to detect both delamination and debonding. Temperature differences indicative of defects, such as a thin delamination heating faster than the thicker, sound pavement around it, are detected by scanners and recorded on videotape. Often, real-image video recording equipment is mounted together with the scanner to record surface defects such as potholes and patches which may otherwise be interpreted incorrectly when viewed on the infrared output. The infrared scanning equipment can be van mounted and operated at speeds of 15 mph.

The major advantages of infrared thermography are its speed and accuracy, relative to destructive methods such as coring, for subsurface data collection. Disadvantages of infrared thermography include its sensitivity to non-pavement-related conditions such as time of day and recent weather conditions. Also, the two-dimensional output cannot indicate the depth of the distressed area. Perhaps the greatest practical disadvantage of infrared, however, is the complexity of the infrared outputs and video images.

Wave Propagation/Spectral Analysis

Wave propagation is a technique for monitoring the dispersion (change in velocity with frequency or wavelength) of surface waves in a pavement, to predict pavement condition. In a layered system, the dispersion of surface waves is indicative of the relative stiffnesses of distinct layers.

Surface waves may be produced using drop weight devices, vibratory devices, or strike hammers. This third method is employed in the testing technique known as spectral analysis of surface waves (SASW). This technique involves using a series of progressively larger hammers to produce waves of increasing wavelength, which tend to propagate through the deeper layers of a pavement. The waves generated in the pavement by the strike hammers, and their dispersions, are monitored by two transducers acting as receivers. The data are collected by a spectral signal analyzer and passed to a computer for processing. The wave velocities can be transformed into representations of modulus versus depth.

Spectral analysis of surface waves has been applied to asphalt, concrete, and asphalt-overlaid concrete pavements, over both fine-grained and coarse-grained subgrades. SASW analysis results have been shown to compare well with backcalculation results from deflection analysis.

An advantage of SASW over deflection-based backcalculation is that it is capable of predicting pavement layer moduli without advance knowledge of layer thicknesses or material types. A disadvantage of SASW is the difficulty and time involved in data collection and interpretation. More automated data acquisition and processing methods are needed to make this testing technique more practical.[50]

Sonic/Ultrasonic/Seismic Wave Analysis

The use of sonic, ultrasonic, and seismic waves to evaluate internal concrete conditions has also been applied to pavements. These techniques involve emission of stress waves from a source (a transducer or high-speed, low-mass projectile) at the pavement surface, and detection of direct or refracted wave characteristics by very precise sensors. Compression and shear waves are used to determine modulus and strength. Horizontal and seismic waves are used to detect voids beneath a concrete slab. Analysis of the reflected wave data can provide information on pavement layer thicknesses and delaminations.[50] The usefulness, speed, accuracy, advantages and disadvantages sonic/ultrasonic/seismic wave analysis for pavement evaluation are not as well established as they are for ground-penetrating radar, infrared thermography, and spectral analysis of surface waves.

Chapter 3

Guidelines for Project-Level Pavement Evaluation

The purpose of project-level pavement evaluation is to assess the current condition of the pavement, identify the key types of deterioration present, identify deficiencies that must be addressed by rehabilitation, and identify uniform sections for rehabilitation design and construction over the project length.

Distress Evaluation

Rehabilitation of a pavement is most likely to be successful – that is, provide satisfactory performance and cost-effectiveness – if it is selected on the basis of knowledge of the types of distresses occurring in the pavement, and understanding of the causes for those distresses. The principal distresses that occur in asphalt, concrete, and asphalt-overlaid concrete pavements, and the mechanisms that cause them, were summarized previously in Tables 1, 2, and 3. More detailed descriptions of these distresses and their causes are given in Appendix A. Many distresses have more than one possible cause. It is important to study the distresses observed in the field survey in order to correctly identify the one or more mechanisms causing the distress observed.

The well-known Pavement Condition Index (PCI) procedure for calculation of a numerical index of pavement condition from distress data, on a scale of 0 to 100, was developed by the U.S. Army Corps of Engineers for application to airfields and to local roads and streets (i.e., military bases).[55] The Corps of Engineers' PCI procedure does not address highway pavements. Some State DOTs, such as Ohio[51] and Washington[52], have developed highway pavement condition index procedures modeled on the PCI procedure. PCI-type procedures, however, are more useful in network-level pavement management – i.e., rehabilitation programming – than for project-level rehabilitation strategy selection and rehabilitation design.

Structural Evaluation

Structural evaluation involves examination of the collected distress, deflection, materials, soils, and drainage information for the following purposes:

- Assessment of the current structural condition of the pavement, that is, how much structural damage has been done to the pavement so far; and
- Assessment of the remaining structural life of the pavement, that is, how many more loadings it can support before failure.

The results of a structural evaluation are also used in dividing a project into structurally uniform sections, identifying areas requiring localized repair, selecting one or more appropriate alternatives for structural improvement, and developing preliminary designs for these alternatives.

Asphalt Pavement Structural Evaluation

Structural evaluation of asphalt pavements may be accomplished using condition data only, deflection measurements only, condition plus deflection data, or traffic data only.

As a general rule, an asphalt pavement is considered to require a structural improvement when 50 percent of the wheelpath area (equivalent to about 10 percent of the total area) of the outer traffic lane has medium- to high-severity alligator cracking.[53] A critical rutting level of one half inch is often cited as indicative of a need for structural improvement. However, rutting may have causes related not only to the load-bearing capacity of the pavement layers, but rather the stability of the mix, so the cause of rutting should be examined before deciding whether or not a structural improvement is the appropriate remedy.

An example of asphalt pavement structural evaluation using condition data is the 1993 AASHTO Guide´s condition method for overlay design of asphalt pavements, also called the component analysis method.[1] This approach involves calculating an “effective Structural Number” using pavement layer material structural coefficients which are less than or equal to those which would be assigned to new materials, depending on the types, extents, and severities of load-related distress present.

Examples of deflection-based approaches to structural evaluation of asphalt pavements are the 1993 AASHTO Guide´s deflection method for overlay design,[1] the Asphalt Institute´s overlay design procedure,[4] and Thompson´s ε-AUPP algorithm,[54] in which asphalt pavement strain (ε) is predicted as a function of area under the pavement profile (AUPP), a deflection basin curvature parameter. Deflection analysis may be used in combination with condition-based overlay design, either solely for the purpose of estimating the resilient modulus of the subgrade soil (e.g., the 1993 AASHTO Guide method), or for the purpose of backcalculating the elastic moduli of other pavement layers, and using these elastic moduli in mechanistic-empirical models for fatigue, rutting, and thermal cracking.

A traffic-based approach to structural design of asphalt pavements is the 1993 AASHTO Guide´s remaining life method of overlay design, in which the structural condition of the existing pavement is determined as a function of the ratio of past ESALs to allowable ESALs. This approach to structural capacity determination has some significant limitations, as discussed in the 1993 AASHTO Guide.

Concrete Pavement Structural Evaluation

Structural evaluation of concrete pavements may be accomplished using condition data only, condition plus deflection data, or traffic data only. An example of concrete pavement structural evaluation using condition data is the 1993 AASHTO Guide´s condition method for overlay design of concrete pavements.[1] This approach involves determining an "effective slab thickness" which is less than or equal to the actual slab thickness, depending on the types, extents, and severities of load-related distress present.

As a general rule, a jointed plain concrete pavement is considered to require a structural improvement when 10 percent of the slabs in the outer traffic lane are cracked.[53] In jointed plain concrete pavement, linear cracking (transverse, longitudinal, diagonal, and corner breaking) of all severities is considered structural distress.

As a general rule, a jointed reinforced concrete pavement is considered to require a structural improvement when 50 percent of the joints in the outer lane have medium- or high-severity joint deterioration, and/or when there are about 75 or more medium- or high-severity transverse cracks per mile in the outer traffic lane.[53] Low-severity transverse cracks are not considered structural distress.

As a general rule, continuously reinforced concrete pavement is considered to require a structural improvement when 10 or more punchouts, steel ruptures, and/or failed patches per mile are present in the outer traffic lane.[53]

No purely deflection-based approaches to structural evaluation exist for concrete pavements, but some methods exist for using deflection analysis in characterizing the structural condition of a concrete slab. One such example is Rolling´s approach[26] to assigning an effective modulus to the concrete slab as a function of its Structural Condition Index (SCI), which is the structural component of the Pavement Condition Index (PCI) derived from the rating system developed by Shahin et al.[55]

Structural evaluation of concrete pavements may be accomplished using deflection analysis in conjunction with condition survey results. An example is the method developed by Hall et al.[50] to assign a qualitative rating to the structural integrity of a concrete slab, based on the mean backcalculated concrete elastic modulus, the percentage of backcalculated modulus values less than 2 million psi, the extent and severity of durability problems, the percentage of the outer traffic lane area which has been repaired, and the percentage of the outer traffic lane area needing new repairs.

Deflection analysis may also be used in combination with condition-based structural characterization of concrete pavements for the purpose of estimating inputs to overlay design (e.g., modulus of subgrade reaction, elastic modulus of the concrete, etc., as in the 1993 AASHTO Guide method), or for the purpose of backcalculating the subgrade k value and concrete slab modulus, and using these elastic moduli in mechanistic-empirical distress models.

A traffic-based approach to structural evaluation of concrete pavements is the 1993 AASHTO Guide´s remaining life method of overlay design, in which the structural condition of the existing pavement is determined as a function of the ratio of past ESALs to allowable ESALs. This approach to structural capacity determination has some significant limitations, as discussed in Part III, Chapter 5 of the 1993 AASHTO Guide.[1]

Asphalt-Overlaid Concrete Pavement Structural Evaluation

Structural evaluation of asphalt-overlaid concrete pavements may be accomplished using condition data only, or condition plus deflection data. An example of asphalt-overlaid concrete pavement structural evaluation using condition data is the 1993 AASHTO Guide´s condition method for overlay design of asphalt-overlaid concrete pavements.[1] This approach involves determining an "effective slab thickness," depending on the types, extents, and severities of load-related distress present.

As a general rule, an asphalt-overlaid jointed concrete pavement (plain or reinforced) is considered to require a structural improvement when it has 75 or more medium- or high-severity reflected cracks, joints, and or patches per mile in the outer traffic lane.[19] An asphalt-overlaid continuously reinforced concrete pavement is considered to require a structural improvement when it has 10 medium- or high-severity reflected cracks, punchouts, or failed patches per mile in the outer traffic lane.[19]

No purely deflection-based approaches to structural evaluation exist for asphalt-overlaid concrete pavements. Structural evaluation of asphalt-overlaid concrete pavements may be accomplished using deflection analysis in conjunction with condition survey results. An example is the method developed by Hall et al.[50] to assign a qualitative rating to the structural integrity of a concrete slab, based on the mean backcalculated concrete elastic modulus, the percentage of backcalculated modulus values less than 2 million psi, the extent and severity of durability problems, the percentage of the outer traffic lane area which has been repaired, and the percentage of the outer traffic lane area needing new repairs.

No purely traffic-based approach exists for structural evaluation of asphalt-overlaid concrete pavements. The traffic-based remaining life methods provided for asphalt and bare concrete pavements in the 1993 AASHTO Guide are not directly applicable to existing asphalt-overlaid concrete pavements, because the original AASHO Road Test performance models are not directly applicable to this type of pavement.

Functional Evaluation

Functional evaluation involves comparing the pavement´s measured roughness, skid resistance, and rut depth (in the cases of asphalt and asphalt-overlaid concrete pavements) to the agency's standards for these functional parameters.

The 1993 AASHTO Guide recommends that, for the purposes of pavement design, the minimum allowable serviceability be selected as a function of the location (urban, rural) and functional class of the roadway. Hall et al. [56] recommend minimum serviceabilty levels of 3.0, 2.5, and 2.0, for ADT levels greater than 10,000, between 3,000 and 10,000, and less than 3,000 respectively. The American Concrete Pavement Association[57] recommends the same trigger levels and identifies the corresponding California Profilograph profile index levels as 60, 80, and 100 respectively. The Pennsylvania DOT[70] recommends minimum serviceability levels of 3.0, 2.5, and 2.0 for Interstates/major arterials, minor arterials/collectors, and local access highways respectively.

As a general rule, faulting is considered to require correction in concrete pavements when it reaches an average level of 0.125 inch in jointed plain concrete pavement or 0.25 inch in jointed reinforced concrete pavement. For the purpose of assessing whether faulting has yet reached an unacceptable level, faulting measured at both joints and transverse cracks should be included in calculating the average.

Drainage Evaluation

The positive subsurface drainage features (edgedrains and outlets and/or a permeable base layer), if any, that a pavement has are identified in the pavement section inventory and field survey. Detailed examples of evaluation of the design adequacy of subsurface drainage features are provided by Wyatt and Macari.[58] The evaluation of the adequacy of the existing subdrainage features may be assessed by examining the following factors:

- What is the expected moisture inflow into the pavement's structure?
- What is the capacity of the pavement base to hold this inflow?
- If the base expected inflow exceeds the base capacity, what outlet features (longitudinal pipes, outlets, daylighting) are provided to remove the excess inflow?
- Are the drainage system features (base permeability, pipe and outlet design, etc.) adequate to permit the removal of the excess inflow from the pavment structure?

The Moisture-Accelerated Distress (MAD) Index method developed by Carpenter et al.[59] is a procedure for rating the potential for moisture-accelerated damage in a pavement, and relating this damage potential to the need for subsurface drainage. The factors considered in calculating a pavement´s MAD index are the climatic region (as defined by temperature and moisture criteria), seasonal moisture concentration, the drainage quality of the base material, and the drainage characteristics of the natural subgrade soil. A pavement with a low MAD index has a high potential for moisture-accelerated damage, and thus has a greater need for subsurface drainage.

Another approach to evaluating the overall quality of drainage is one which involves evaluation of the drainability of the base together with the drainability of the subgrade soil.[60] The drainability of the base (assuming an impermeable subgrade) is first assessed as a functioning of the following parameters:

- Permeability, which is the measure of how rapidly water can move through a material. Permeability is a function of the percentage and type of fines, effective grain size, specific gravity of the solids, and the dry density of the base material.

- Effective porosity, which is the measure of the material's ability to hold water. Effective porosity is a function of dry density, percent fines, and specific gravity of the solids.

- Drainage time, which is the time required to drain a 100 percent saturated base layer to some lower degree of saturation. Drainage time to a given saturation level is a function of permeability, effective porosity, and base layer geometry (thickness, width, longitudinal grade, and transverse grade).

For highway pavements, base drainability is assessed qualitatively depending on the time required to drain a fully saturated base to 85 percent saturation. Drainage times of about 5 hours are less are considered satisfactory, drainage times between about 5 hours and 12 hours are considered marginal, and drainage times in excess of about 12 hours are considered unsatisfactory.

The drainability of the subgrade soil is assessed using the Natural Drainage Index, a parameter used to characterize soils for agricultural purposes.[59] Natural Drainage Index values for soils located in the vicinity of the pavement being evaluated may be obtained from county soil reports. The Natural Drainage Index is a useful quantitative parameter, but its interpretation for agricultural purposes (for which soils should ideally neither drain too much nor too little) is different than its interpretation for pavement evaluation purposes (for which soils should ideally drain as much as possible).

Soils with NDI values between –10 and –2 are considered to have good subgrade drainability from a pavement evaluation standpoint. Soils with NDI values between –2 and +2.5 are considered to have average subgrade drainability from a pavement evaluation standpoint. Soils with NDI values between +2.5 and +10 are also considered to have poor subgrade drainability from a pavement evaluation standpoint.

Base drainability and subgrade drainability may be considered together in assessing the overall quality of drainage, using Table 5.

Table 5. Assessment of overall quality of drainage.[60]

		Base drainability		
		Satisfactory	Marginal	Unsatisfactory
Subgrade soil drainability	Good	Excellent	Good	Fair to poor
	Fair	Good	Fair	Poor to very poor
	Poor	Fair to poor	Poor to very poor	Very poor

Identification of Uniform Sections

Sections within the project that are uniform with respect to design, geometry, materials, structural capacity, soils, distress, traffic, drainage, etc., should be identified on the basis of the collected inventory, materials, distress, deflection, and other data.

Appendix J of the 1993 AASHTO Guide[1] presents a method for delineating pavement sections which are statistically homogeneous with respect to one parameter, e.g., maximum deflection. The simultaneous consideration of several inventory, distress, and deflection parameters could conceivably result in the division of the project into several short sections.

The shortest section length for which rehabilitation can realistically be designed and constructed, e.g., one half mile, should be determined. Uniform sections should be combined as necessary to make rehabilitation design sections of at least this minimum length, and the representative conditions over each of these sections should be quantified.

Chapter 4

Guidelines for Selection of Rehabilitation Techniques

The purpose of rehabilitation technique selection is to identify candidate rehabilitation techniques which are best suited to the correction of existing distress and achievement of desired improvements in the structural capacity, functional adequacy, and drainage adequacy of the pavement.

Pavement Rehabilitation Treatments

A pavement rehabilitation strategy is a combination of individual rehabilitation treatments. A rehabilitation strategy is explicit enough, in terms of both the types and quantities of treatments to be applied, that it can be evaluated and compared with other rehabilitation strategy alternatives, in terms of expected performance and costs. For example, "resurfacing" is not a sufficiently detailed description of a rehabilitation strategy, nor is "asphalt overlay." On the other hand, "2- to 3-inch overlay of both traffic lanes and shoulders, after 3 percent slab replacement in the outer lane" is a sufficiently explicit description of the proposed work to be evaluated as a rehabilitation strategy.

The different rehabilitation treatments which may be used as part of a rehabilitation strategy are briefly described below. Detailed descriptions of each of these treatments are provided in Appendix B of this Guide. The partial-depth patching and surface improvement techniques described for asphalt pavements also apply to asphalt-overlaid concrete pavements. Full-depth patching and most of the overlay techniques described for concrete pavements also apply to asphalt-overlaid concrete pavements.

Asphalt Pavement Rehabilitation Treatments

Full-depth or partial-depth repair, or **patching**, of an asphalt pavement is localized repair of distresses related to structural damage, materials problems, or construction problems. The repair may be full depth (down to the subgrade or an intact subbase layer) or partial depth (asphalt surface only), depending on the nature of the distress. Patching may be done on an asphalt pavement either for maintenance purposes or rehabilitation purposes. Maintenance patching is expedient and temporary repair, often done in cold weather, with cold-mixed patching mixtures. As a result, maintenance patches generally do not exhibit the same long-term stability and durability as hot-mixed patching mixtures, carefully constructed as part of a rehabilitation strategy. Patching for rehabilitation purposes may be done as part of a restoration of the existing pavement structure, or may be done in preparation for resurfacing.

Cold milling is the removal of material from an asphalt pavement surface, using carbide bits mounted on a rotating drum. Cold milling may be done for one or more of several reasons: to texturize the surface prior to resurfacing in order to enhance bond, to remove excess asphalt concrete thickness, to remove oxidation at the surface, to remove unstable asphalt concrete material, to modify the longitudinal and/or transverse grade, to maintain or reestablish curb and gutter lines prior to resurfacing, to remove rutting, and/or to remove bumps. Infrequently, cold milling is done without subsequent resurfacing, to remove rutting and/or bumps, and/or to improve surface friction.

Hot in-place recycling is the on-site rejuvenation of aged asphalt concrete material. Hot in-place recycling is usually but not always done in conjunction with resurfacing. Rejuvenating the existing surface prior to placing an overlay enhances bond and discourages reflection cracking. Hot in-place recycling may also be done without a subsequent overlay, to correct surface distresses such as minor corrugations or bleeding. The process involves heating the surface to the desired depth with slow-moving, high-intensity heaters, possibly mixing in a rejuvenating agent (if the asphalt cement is very brittle) and/or virgin asphalt concrete material, and then either compacting the rejuvenated surface, or placing an overlay.

Cold in-place recycling is the on-site cold milling of asphalt concrete material, mixing of the material with an emulsified asphalt and/or other additives (lime is often used), and laying down and recompacting the material. Cold in-place recycled material is not as stiff or as stable as hot-mix asphalt, so it usually must be capped with an asphalt concrete wearing course or a single or double surface treatment.

An **asphalt overlay** of an asphalt pavement may be placed to improve ride quality and/or surface friction, or may be placed for the purpose of substantially increasing structural capacity. The two most commonly used approaches to structural design of asphalt overlays of asphalt pavements are (1) the structural deficiency approach, exemplified by the 1993 AASHTO[1] procedure; and (2) the deflection-based approach, exemplified by the Asphalt Institute[4] procedure. Much less common is the mechanistic approach, in which fatigue and rutting performance are predicted using mechanistic-empirical models. The performance of an asphalt overlay depends primarily on the thickness of the overlay, its asphalt concrete mix design, and the type and extent of preoverlay repair and surface preparation.

A **concrete overlay**, or **whitetopping**, of an asphalt pavement is usually done to increase structural capacity. Conventional concrete overlays are constructed with normal Portland cement concrete mixtures and paving methods. The structural design of conventional concrete overlays is comparable to that of new concrete pavements or unbonded concrete overlays. Ultrathin concrete overlays are constructed with very short joint spacings, and sometimes with high-early-strength mixes and fast-track paving methods, with the goal of opening the overlay to traffic quickly. The design and construction methods for ultrathin whitetopping are relatively new and in many respects are still in development.

Concrete Pavement Rehabilitation Treatments

Full-depth repair of a concrete or asphalt-overlaid concrete pavement is localized repair of distresses related to structural damage, materials problems, or construction problems. Full-depth repairs are constructed across joints and between joints, across the full lane width, and at least 6 ft long. In short-jointed plain concrete pavements, **slab replacement** may cost less than repair of a portion of a slab. Full-depth repairs in jointed concrete pavements should be dowelled at the transverse joints to the adjacent slabs. The load transfer system (size, number, and layout of the dowel bars) should be selected considering the truck traffic using the pavement. Full-depth repairs in continuously reinforced concrete pavements must be continuously reinforced as well, with the steel tied or welded at the transverse joints to the steel in the adjacent slabs.

Full-depth repairs in both jointed and continuously reinforced concrete pavements should be separated from the adjacent lane slab by a bondbreaker along the longitudinal joint. Full-depth repairs in asphalt-overlaid concrete pavements should be constructed of concrete capped with asphalt. The concrete portion of the repair should be dowelled or tied as appropriate for the type of concrete pavement present. Full-depth repairs may be constructed with normal paving concrete mixtures, or with high-early-strength mixtures, if quick opening to traffic is necessary. Full-depth repairs may be done as part of a restoration of the existing pavement structure, or may be done in preparation for resurfacing.

Partial-depth repair of a concrete pavement is localized repair of distresses which are confined to the upper third of the slab, such as joint spalling. Partial-depth repairs are not appropriate for distresses which extend the full thickness of the slab, or to the depth of reinforcing steel or dowel or tie bars, if present. Partial-depth repairs may be constructed with normal paving concrete mixtures, or with high-early-strength mixtures or specialty repair materials, if quick opening to traffic is necessary. Partial-depth repairs may be done as part of a restoration of the existing pavement structure, or may be done in preparation for resurfacing. Partial-depth distresses are often repaired with asphalt when the overlay to be placed will be asphalt or unbonded concrete. Partial-depth distresses should be repaired with concrete if the overlay to be placed will be bonded concrete.

Undersealing, also called **subsealing** or **slab stabilization,** is the filling of localized voids under slab corners by injection of a filler material, in fluid state, through holes drilled through the slab. The purpose of undersealing is to reduce corner deflections and thereby reduce stresses in the slab, and possibly reduce faulting development as well. The most commonly used filler is cement grout, although asphalt cement has been used as well. Undersealing differs from **slabjacking**, which is done with the same equipment and materials, in that undersealing fills voids without raising the slab, whereas slabjacking raises slabs. Undersealing should only be done at slab corners which have voids; the practice of "blanket undersealing" of all corners, whether they have voids or not, can disrupt the uniformity of support and increase stresses in the concrete slab.

Load transfer restoration is the installation of load transfer devices (either dowel bars or other devices which have been developed for this purpose) across undowelled joints and/or cracks. Load transfer restoration may be done as part of a restoration of the existing pavement structure, or may be done in preparation for resurfacing. Although it is a restoration technique, it could be argued that load transfer restoration increases structural capacity, because it increases load transfer and thereby decreases corner stresses in the adjacent slabs. The process involves sawing slots or drilling core holes across the joint or crack, installing the dowel bars on chairs or load transfer devices into the core holes, and backfilling, usually with the same type of material that would be used for partial-depth repairs. The load transfer system (size, number, and layout of the dowels or load transfer devices) should be selected considering the truck traffic using the pavement. Load transfer restoration requires considerable care in construction, but recent advances in equipment and methods have greatly increased the quality and efficiency of the operation.

Joint resealing is generally considered a maintenance activity, but may also be done in conjunction with other restoration techniques for rehabilitation purposes. The process involves removing the old sealant if present, sawing a new joint reservoir of appropriate dimensions for the sealant to be used, thorough cleaning of the new reservoir, installing the sealant, and for some sealant types, tooling it into place. Material used for joint resealing include rubberized asphalt, silicone, and preformed neoprene inserts. When done as part of a restoration effort, joint resealing should be done after all other treatments, e.g., full-depth repair, partial-depth repair, undersealing, load transfer restoration, and/or diamond grinding.

Diamond grinding is the removal of a shallow depth of material from a concrete pavement surface, using diamond saw blades closely spaced on a rotating drum. Diamond grinding is most frequently done to remove faulting at joints and cracks, or to remove bumps. In addition to improving the ride quality of the pavement, diamond grinding improves surface texture and friction.

Grooving is also done using diamond saw blades, but the blades are space farther apart than they are for diamond grinding. The purpose of grooving is to improve wet weather surface friction. On highway pavements it is usually done on curves and ramps which have inadequate cross slope or which have surfaces which have become polished, but which, perhaps because of geometric constraints, cannot easily be resurfaced.

Pressure relief joints are full-depth asphalt patches or other compressible materials installed at intervals of a few hundred feet, in pavements which are at risk of joint blowups due to compressive stress buildup in the slab. Pressure relief joints have been overused in the past, constructed in pavements in which they were not really needed, and in many cases have done much more harm than good, causing nearby joints to open and fault excessively. Pressure relief joints may be appropriate, however, for jointed pavements with reactive aggregate, or under certain climatic conditions, for long-jointed pavements.

An **asphalt overlay** of a concrete pavement or asphalt-overlaid concrete pavement may be placed to improve ride quality and/or surface friction, and/or or may be placed for the purpose of substantially increasing structural capacity. The most commonly used approach to structural design of asphalt overlays of concrete pavements and asphalt-overlaid concrete pavements is the structural deficiency approach, exemplified by the 1993 AASHTO procedure. The performance of an asphalt overlay depends primarily on the applied traffic loads, the thickness of the overlay, its asphalt concrete mix design, and the type and extent of preoverlay repair and surface preparation.

An **asphalt or concrete overlay of a fractured concrete pavement** is placed to increase structural capacity. Slab fracturing may be done for two reasons: to attempt to mitigate reflection cracking in the overlay, and/or to dispense with preoverlay repair of a concrete pavement with extensive slab cracking and or materials-related deterioration. Jointed plain concrete pavements are **cracked and seated**, meaning that the slab is cracked into pieces between about 1 and 3 feet on a side, and seated with a heavy roller. Jointed reinforced concrete pavements are **broken and seated**, meaning that the slab is broken and the reinforcing steel is ruptured (this may require greater impact force than cracking an unreinforced slab), prior to seating with a heavy roller. Jointed plain, jointed reinforced, and continuously reinforced pavements may be **rubblized**, meaning that the slab is pulverized into pieces no more than about 6 inches. Structural design for an asphalt or concrete overlay of a cracked and seated or broken and seated concrete may be done using either flexible pavement or rigid pavement overlay design methods. Structural design for an asphalt or concrete overlay of a rubblized pavement is similar to that for a new asphalt or concrete pavement on a high-strength granular base.

A **bonded concrete overlay** of a concrete pavement may be placed to increase strucrural capacity or to increase serviceability of an otherwise structurally sound concrete pavement. Structural design of bonded concrete overlays is usually done by the Corps of Engineers method, a structural deficiency approach which is also used in the 1993 AASHTO Guide. Construction of a bonded concrete overlay requires careful preparation of the surface to ensure a strong bond between the existing slab and the overlay. Bonded concrete overlays are not used often, because they perform best on pavements in good to fair condition, that is, pavements which are not in urgent need of rehabilitation. The sensitivity of a bonded concrete overlay to underlying pavement condition necessitates exhaustive repair, which is generally not cost-effective (in comparison with other resurfacing options) for pavements with substantial slab cracking and/or joint deterioration.

An **unbonded concrete overlay** of a concrete or asphalt-overlaid concrete pavement is placed to increase structural capacity. An unbonded concrete overlay is an attractive alternative to reconstruction when construction duration is a pressing issue (e.g., for high traffic volumes and/or very poor subgrade conditions). Asphalt concrete is the preferred and most effective material for separating the overlay form the old pavement. Other materials that have been used in the past, less widely and/or with less success, include permeable asphalt-treated gravel, sand asphalt mixtures, unstabilized granular materials, and double-layered polyethylene sheeting. Unbonded concrete overlays require little or no preoverlay repair, and are thus well suited to badly deteriorated concrete pavements.

Structural design of an unbonded concrete overlay has traditionally been done using the Corps of Engineers method. An alternative approach to design of an unbonded overlay is to design it as if it were a new pavement on a rigid base. In jointed unbonded concrete overlays, the joints should be spaced more closely than they would be in a new pavement on a granular base, and the overlay´s transverse joints and the old pavement´s transverse joints should be mismatched. An unbonded concrete overlay may also be continuously reinforced.

Subdrainage improvement, for any type of pavement, may involve such activities as installation of longitudinal subdrains and outlets alongside an existing pavement structure, or daylighting a base layer by replacing base material under the shoulders with better-draining material. Whether or not retrofit subdrainage improvements are beneficial to the performance of the existing pavement depends on whether or not water in the pavement structure can be effectively removed, and how well the subdrainage system is designed, constructed, and maintained.

Techniques Indicated by Structural Evaluation

The results of the structural evaluation in Step 2 will indicate whether or not the rehabilitation strategies to be developed should include some technique for structural improvement, i.e., resurfacing or reconstruction.

The type of structural improvement most likely to perform well and be cost-effective for a given pavement depends on the point in the pavement's structural life at which the structural improvement is made – specifically, the amount of structural distress present. The performance of some types and thicknesses of overlays (e.g., bonded concrete overlays, thinner asphalt overlays) is very sensitive to the extent and quality of preoverlay repairs. For such overlay types, preoverlay repair may be the single largest component of the initial cost of the rehabilitation. The long-term performance of overlays depends primarily on (a) the overlay thickness, (b) how much of the needed preoverlay repair was actually done, and (c) the quality of those repairs. Progressively greater amounts of existing structural distress require the use of thicker overlays and/or overlay types whose performance is less sensitive to the degree of preoverlay repair done.

There are no simple rules or universally accepted distress trigger levels for identifying the type of structural improvement which is most appropriate at a given point in a pavement's life. The decision depends on several factors, including the extent of structural distress present, the extent of preoverlay repair planned (often the available funds are not sufficient to repair 100 percent of the medium- and high-severity distress present), the type of overlay, and the overlay thickness. It is recommended that at least two structural improvement alternatives be considered and developed in detail (thickness design, performance prediction, costs) for any pavement in need of structural improvement.

The progression of structural improvements most likely to perform well and be cost-effective at different points in the life a pavement can be illustrated conceptually as in Figure 19. This progression of pavement rehabilitation needs is typical, but unusual circumstances such as premature failure may necessitate more substantial rehabilitation earlier than is typical. Trigger levels of key distresses indicating a need for structural improvement were identified in the previous step on pavement evaluation.

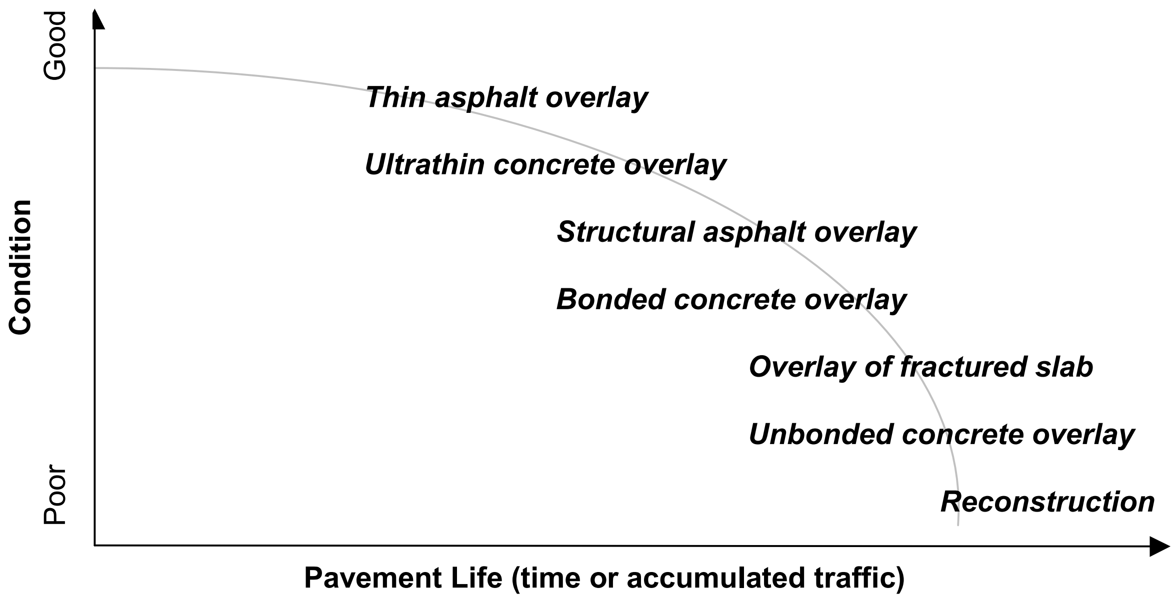

Figure 19. Structural improvement options most suitable at different points in a pavement's life.

The term "thin asphalt overlay" refers here to an asphalt overlay used either as a maintenance treatment or a functional rehabilitation treatment. The term "asphalt overlay" refers here to an asphalt overlay used as a structural rehabilitation treatment.

The term "ultrathin concrete overlay" refers here to a concrete overlay used as a functional rehabilitation treatment for low-volume roads and streets (less than 1 million design ESALs). The terms "bonded concrete overlay" and "unbonded concrete overlay" refer to concrete overlays used as structural rehabilitation treatments.

Not all of the options shown in Figure 19 are applicable to all pavement types. The specific structural improvement options applicable to each pavement type are listed below.

Structural Improvements for Asphalt Pavements

- Structural asphalt overlay
- Conventional oncrete overlay
- Reconstruction (in either asphalt or concrete)

Structural Improvements for Concrete Pavements

- Structural asphalt overlay
- Bonded concrete overlay
- Asphalt or concrete overlay of fractured concrete slab
- Unbonded concrete overlay
- Reconstruction (in either asphalt or concrete)

Structural Improvements for Asphalt-Overlaid Concrete Pavements

- Structural asphalt overlay
- Asphalt or concrete overlay of fractured concrete slab (note: to fracture the existing slab adequately, the existing asphalt overlay should be removed)
- Unbonded concrete overlay
- Reconstruction (in either asphalt or concrete)

Techniques Indicated by Functional Evaluation

All overlay and reconstruction options supersede the need to make other functional improvements to the pavement surface, i.e., to reduce roughness and/or improve wet weather friction. The specific functional improvement options applicable to each pavement type are listed below.

Functional Improvements for Asphalt Concrete Pavements

- Cold milling
- Hot surface recycling
- Thin asphalt overlay
- Ultrathin concrete overlay

There are several other surface improvement options available for asphalt pavements, such as chip seals, slurry seals, microsurfacing, etc. However, these treatments are typically classified as maintenance rather than rehabilitation, and thus are not addressed in this Guide.

Functional Improvements for Concrete Pavements

- Diamond grinding
- Grooving
- Thin asphalt overlay
- Bonded concrete overlay

Functional Improvements for Asphalt-Overlaid Concrete Pavements

- Cold milling
- Hot surface recycling
- Thin asphalt overlay
- Ultrathin concrete overlay

Again, chip seals and other types of surface improvements may be applied to asphalt-overlaid concrete pavements, but these are considered maintenance treatments and thus are not addressed in this Guide.

Techniques Indicated by Distress Evaluation

In addition to the potential needs for structural and/or functional improvement, other rehabilitation techniques may be needed to repair specific distresses. Some of these techniques may be considered either "preoverlay repair" or "restoration," depending on whether or not they are being done in conjunction with an overlay.

At this point in the rehabilitation strategy selection process, the individual rehabilitation techniques that are potentially applicable to the types of distress present are identified. Critical levels indicating the need for correction were identified previously for key distresses. For several distress types, there is more than one rehabilitation technique that may be used to correct or eliminate the distress. The candidate rehabilitation techniques best suited to the treatment of specific distresses in asphalt pavements, concrete pavements, and asphalt-overlaid concrete pavements are indicated in Tables 6, 7, and 8 respectively.

Some individual rehabilitation techniques supercede others, and some combinations of rehabilitation techniques are infeasible, because the techniques cannot or should not be done together. The combining of individual rehabilitation techniques into one or more feasible rehabilitation strategy alternatives is addressed in the next chapter of this Guide.

Techniques Indicated by Drainage Evaluation

The results of the drainage evaluation in the previous chapter will indicate whether or not a subdrainage improvement should be included in the rehabilitation strategy alternatives developed. Subdrainage improvement for either an asphalt or a concrete pavement may involve installation or replacement of longitudinal subdrains and outlets alongside an existing pavement structure, or daylighting a base layer by replacing base material under the shoulders with better-draining material. Cleaning existing longitudinal subdrains and removing soil and vegetation clogging an existing daylighted base layer are generally considered maintenance activities.

Table 6. Rehabilitation techniques best suited for asphalt pavement distresses.

Asphalt Pavement Distresses	Asphalt Pavement Rehabilitation Techniques							
	Full-depth asphalt repair	Partial-depth asphalt repair	Cold milling	Hot or cold in-place recycling	Asphalt overlay	Concrete overlay	Subdrainage improvement	Reconstruction (asphalt or concrete)
Fatigue cracking	✓	✓	✓	✓	✓	✓		✓
Block cracking		✓	✓	✓	✓	✓		✓
Thermal cracking	✓		✓	✓	✓	✓		✓
Longitudinal cracking	✓			✓	✓	✓		✓
Slippage cracking		✓	✓	✓	✓	✓		
Bleeding	✓	✓	✓	✓	✓	✓		
Rutting			✓	✓	✓	✓		✓
Shoving			✓	✓		✓		✓
Weathering		✓	✓	✓	✓	✓		
Ravelling		✓	✓	✓	✓			
Pumping							✓	
Stripping	✓	✓	✓		✓			✓
Pothole	✓	✓		✓				✓
Bumps, settlements, heaves	✓		✓	✓	✓	✓		✓

Table 7. Rehabilitation techniques best suited for concrete pavement distresses.

Concrete Pavement Distresses	Concrete Pavement Rehabilitation Techniques													
	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slab jacking	Load transfer restoration	Joint resealing	Diamond grinding	Grooving	Pressure relief joints	Asphalt overlay	AC overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay	Subdrainage improvement	Reconstruction (asphalt or concrete)
Corner break	✓									✓		✓		✓
Linear cracking	✓									✓		✓		✓
Punchout	✓									✓		✓		✓
“D” cracking	✓								✓	✓		✓		✓
Alkali-aggregate reaction	✓							✓	✓	✓		✓		✓
Map cracking, crazing, scaling		✓							✓					
Joint seal damage					✓									
Joint spalling	✓	✓								✓		✓		✓
Blowup	✓									✓		✓		✓
Pumping			✓	✓									✓	
Faulting				✓		✓			✓	✓	✓	✓		
Bumps, settlements, heaves	✓					✓			✓	✓	✓	✓		✓
Polishing						✓	✓		✓		✓			

Table 8. Rehabilitation techniques best suited for asphalt-overlaid concrete pavement distresses.

Asphalt-Overlaid Concrete Pavement Distresses	Asphalt-Overlaid Concrete Pavement Rehabilitation Treatment								
	Full-depth concrete or composite repair	Partial-depth asphalt repair	Cold milling	Hot surface recycling	Asphalt overlay	Asphalt overlay of fractured concrete slab	Unbonded oncrete overaly	Subdrainage improvement	Reconstruction (asphlat or concrete)
Reflection cracking	✓	✓	✓		✓	✓	✓		✓
Fatigue cracking		✓	✓		✓	✓	✓		✓
Punchout	✓				✓	✓	✓		✓
Rutting			✓	✓	✓				
Shoving and corrugation			✓	✓	✓				
Stripping			✓		✓		✓		✓
Pothole	✓	✓			✓		✓		✓
“D” cracking	✓				✓		✓		✓

Chapter 5

Guidelines for Formation of Rehabilitation Strategies

The purpose of formation of rehabilitation strategies is to combine individual rehabilitation techniques into one or more rehabilitation strategy alternatives, developed in sufficient detail that their performance and costs may confidently estimated.

Combine Rehabilitation Techniques Into Strategies

This involves examining the set of individual rehabilitation techniques identified in the previous step, and identifying one or more ways in which the techniques may be grouped into strategies. This requires recognizing when some techniques supercede others, and when some techniques are incompatible with others. It is recommended that each rehabilitation strategy alternative be developed on the basis of the results of the evaluation of the outer traffic lane. Rehabilitation treatments for adjacent traffic lanes and shoulders should selected to be compatible with the outer traffic lane rehabilitation strategy.

The formation of each rehabilitation strategy alternative should address the following four issues:

1. Is a structural improvement needed to correct a structural deficiency?
2. Is a functional improvement needed to correct a functional deficiency (if present and not corrected by a structural improvement)?
3. What additional repair techniques are needed?
4. Is a drainage improvement needed to correct a drainage deficiency?

Multiple rehabilitation strategy alternatives may be developed by considering more than one structural improvement option, more than one functional improvement option, and/or more than one feasible combination of repair techniques. Variations on the rehabilitation strategy alternatives may be developed by considering different overlay thickness designs and/or different quantities of repair.

Formation of Feasible Rehabilitation Strategies for Asphalt Pavements

1. Correct structural deficiency, if present.

If a structural deficiency exists, each of the rehabilitation strategy alternatives should include one of the following techniques:

- Asphalt overlay
- Concrete overlay
- Reconstruction

2. Correct functional deficiency, if present and not addressed by a structural improvement.

If a functional deficiency exists and is not addressed by a structural improvement, each of the rehabilitation strategy alternatives should include one of the following techniques:

- Cold milling
- Hot surface recycling
- Thin asphalt overlay
- Ultrathin concrete overlay

3. Select additional repair techniques to correct distresses not corrected by a structural or functional improvement.

The feasible combinations of techniques which may be used in nonoverlay or overlay rehabilitation of asphalt pavement are shown in Table 9. For example, rehabilitation strategy alternatives involving asphalt overlay may be formed using some or all of the combinations listed as AC 12 through AC 23. Note that reconstruction is not shown in the table because it does not require combination with any other repair or resurfacing techniques.

4. Select a drainage improvement to correct drainage deficiency, if present.

If a drainage deficiency exists, a drainage improvement option may be considered for inclusion in some or all of the rehabilitation strategy alternatives. Drainage improvement options for in-service pavements may include retrofitting or replacing longitudinal subdrains and outlets, or daylighting the base by replacing shoulder base material and repaving the shoulders.

Table 9. Feasible combinations of techniques for rehabilitation of asphalt pavement.

№	Full-depth asphalt repair	Partial-depth asphalt repair	Cold milling	In-place recycling	Asphalt overlay	Concrete overlay
Nonoverlay combinations:						
AC 1	✓					
AC 2		✓				
AC 3			✓			
AC 4				✓		
AC 5	✓	✓				
AC 6	✓		✓			
AC 7	✓			✓		
AC 8	✓	✓	✓			
AC 9	✓	✓		✓		
AC 10		✓	✓			
AC 11		✓		✓		
Asphalt overlay combinations:						
AC 12					✓	
AC 13	✓				✓	
AC 14		✓			✓	
AC 15			✓		✓	
AC 16				✓	✓	
AC 17	✓	✓			✓	
AC 18	✓		✓		✓	
AC 19	✓			✓	✓	
AC 20	✓	✓	✓		✓	
AC 21	✓	✓		✓	✓	
AC 22		✓	✓		✓	
AC 23		✓		✓	✓	
Concrete overlay combinations:						
AC 24						✓
AC 25	✓					✓
AC 26		✓				✓
AC 27			✓			✓
AC 28				✓		✓
AC 29	✓	✓				✓
AC 30	✓		✓			✓
AC 31	✓			✓		✓
AC 32	✓	✓	✓			✓
AC 33	✓	✓		✓		✓
AC 34		✓	✓			✓
AC 35		✓		✓		✓

These drainage improvement options are presented with the caveat that it is difficult to predict what effects drainage improvement efforts will have on the performance of the rehabilitated pavement. Reconstruction alternatives may of course be developed to include design of a completely new drainage system.

Formation of Feasible Rehabilitation Strategies for Concrete Pavements

1. Correct structural deficiency, if present.

If a structural deficiency exists, each of the rehabilitation strategy alternatives should include one of the following techniques:

- Asphalt overlay
- Bonded concrete overlay
- Asphalt or concrete overlay of fractured slab
- Unbonded concrete overlay
- Reconstruction

2. Correct functional deficiency, if present and not addressed by a structural improvement.

If a functional deficiency exists and is not addressed by a structural improvement, each of the rehabilitation strategy alternatives should include one of the following techniques.

If faulting is excessive, select:

- Diamond grinding

If faulting is not excessive but skid resistance is inadequate, select one of the following:

- Diamond grinding
- Grooving

3. **Select additional repair techniques to correct distresses not corrected by a structural or functional improvement.**

The feasible combinations of techniques which may be used in nonoverlay or overlay rehabilitation of concrete pavement are shown in Table 10. For example, rehabilitation strategy alternatives involving asphalt overlay may be formed using some or all of the combinations listed as PCC 144 through PCC 167. Note that reconstruction is not shown in the table because it does not require combination with any other repair or resurfacing techniques.

4. **Select a drainage improvement to correct drainage deficiency, if present.**

If a drainage deficiency exists, a drainage improvement option may be considered for inclusion in some or all of the rehabilitation strategy alternatives. Drainage improvement options for in-service pavements may include retrofitting or replacing longitudinal subdrains and outlets, or daylighting the base by replacing shoulder base material and repaving the shoulders.

These drainage improvement options are presented with the caveat that it is difficult to predict what effects drainage improvement efforts will have on the performance of the rehabilitated pavement. Reconstruction alternatives may of course be developed to include design of a completely new drainage system.

Table 10. Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or cncrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
Nonoverlay combinations:												
PCC 1	✓											
PCC 2		✓										
PCC 3	✓	✓										
PCC 4			✓									
PCC 5	✓		✓									
PCC 6		✓	✓									
PCC 7	✓	✓	✓									
PCC 8				✓								
PCC 9	✓			✓								
PCC 10		✓		✓								
PCC 11	✓	✓		✓								
PCC 12			✓	✓								
PCC 13	✓		✓	✓								
PCC 14		✓	✓	✓								
PCC 15	✓	✓	✓	✓								
PCC 16					✓							
PCC 17	✓				✓							
PCC 18		✓			✓							
PCC 19	✓	✓			✓							
PCC 20			✓		✓							
PCC 21	✓		✓		✓							
PCC 22		✓	✓		✓							
PCC 23	✓	✓	✓		✓							

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or cncrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
PCC 24				✓	✓							
PCC 25	✓			✓	✓							
PCC 26		✓		✓	✓							
PCC 27	✓	✓		✓	✓							
PCC 28			✓	✓	✓							
PCC 29	✓		✓	✓	✓							
PCC 30		✓	✓	✓	✓							
PCC 31	✓	✓	✓	✓	✓							
PCC 32						✓						
PCC 33	✓					✓						
PCC 34		✓				✓						
PCC 35	✓	✓				✓						
PCC 36			✓			✓						
PCC 37	✓		✓			✓						
PCC 38		✓	✓			✓						
PCC 39	✓	✓	✓			✓						
PCC 40				✓		✓						
PCC 41	✓			✓		✓						
PCC 42		✓		✓		✓						
PCC 43	✓	✓		✓		✓						
PCC 44			✓	✓		✓						
PCC 45	✓		✓	✓		✓						
PCC 46		✓	✓	✓		✓						
PCC 47	✓	✓	✓	✓		✓						

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
PCC 48					✓	✓						
PCC 49	✓				✓	✓						
PCC 50		✓			✓	✓						
PCC 51	✓	✓			✓	✓						
PCC 52			✓		✓	✓						
PCC 53	✓		✓		✓	✓						
PCC 54		✓	✓		✓	✓						
PCC 55	✓	✓	✓		✓	✓						
PCC 56				✓	✓	✓						
PCC 57	✓			✓	✓	✓						
PCC 58		✓		✓	✓	✓						
PCC 59	✓	✓		✓	✓	✓						
PCC 60			✓	✓	✓	✓						
PCC 61	✓		✓	✓	✓	✓						
PCC 62		✓	✓	✓	✓	✓						
PCC 63	✓	✓	✓	✓	✓	✓						
PCC 64							✓					
PCC 65	✓						✓					
PCC 66		✓					✓					
PCC 67	✓	✓					✓					
PCC 68			✓				✓					
PCC 69	✓		✓				✓					
PCC 70		✓	✓				✓					
PCC 71	✓	✓	✓				✓					

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
PCC 72				✓			✓					
PCC 73	✓			✓			✓					
PCC 74		✓		✓			✓					
PCC 75	✓	✓		✓			✓					
PCC 76			✓	✓			✓					
PCC 77	✓		✓	✓			✓					
PCC 78		✓	✓	✓			✓					
PCC 79	✓	✓	✓	✓			✓					
PCC 80					✓		✓					
PCC 81	✓				✓		✓					
PCC 82		✓			✓		✓					
PCC 83	✓	✓			✓		✓					
PCC 84			✓		✓		✓					
PCC 85	✓		✓		✓		✓					
PCC 86		✓	✓		✓		✓					
PCC 87	✓	✓	✓		✓		✓					
PCC 88				✓	✓		✓					
PCC 89	✓			✓	✓		✓					
PCC 90		✓		✓	✓		✓					
PCC 91	✓	✓		✓	✓		✓					
PCC 92			✓	✓	✓		✓					
PCC 93	✓		✓	✓	✓		✓					
PCC 94		✓	✓	✓	✓		✓					
PCC 95	✓	✓	✓	✓	✓		✓					

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
PCC 96								✓				
PCC 97		✓						✓				
PCC 98			✓					✓				
PCC 99		✓	✓					✓				
PCC 100				✓				✓				
PCC 101		✓		✓				✓				
PCC 102			✓	✓				✓				
PCC 103		✓	✓	✓				✓				
PCC 104					✓			✓				
PCC 105		✓			✓			✓				
PCC 106			✓		✓			✓				
PCC 107		✓	✓		✓			✓				
PCC 108				✓	✓			✓				
PCC 109		✓		✓	✓			✓				
PCC 110			✓	✓	✓			✓				
PCC 111		✓	✓	✓	✓			✓				
PCC 112						✓		✓				
PCC 113		✓				✓		✓				
PCC 114			✓			✓		✓				
PCC 115		✓	✓			✓		✓				
PCC 116				✓		✓		✓				
PCC 117		✓		✓		✓		✓				
PCC 118			✓	✓		✓		✓				
PCC 119		✓	✓	✓		✓		✓				

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
PCC 120					✓	✓		✓				
PCC 121		✓			✓	✓		✓				
PCC 122			✓		✓	✓		✓				
PCC 123		✓	✓		✓	✓		✓				
PCC 124				✓	✓	✓		✓				
PCC 125		✓		✓	✓	✓		✓				
PCC 126			✓	✓	✓	✓		✓				
PCC 127		✓	✓	✓	✓	✓		✓				
PCC 128							✓	✓				
PCC 129		✓					✓	✓				
PCC 130			✓				✓	✓				
PCC 131		✓	✓				✓	✓				
PCC 132				✓			✓	✓				
PCC 133		✓		✓			✓	✓				
PCC 134			✓	✓			✓	✓				
PCC 135		✓	✓	✓			✓	✓				
PCC 136					✓		✓	✓				
PCC 137		✓			✓		✓	✓				
PCC 138			✓		✓		✓	✓				
PCC 139		✓	✓		✓		✓	✓				
PCC 140				✓	✓		✓	✓				
PCC 141		✓		✓	✓		✓	✓				
PCC 142			✓	✓	✓		✓	✓				
PCC 143		✓	✓	✓	✓		✓	✓				

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
Asphalt overlay combinations:												
PCC 144									✓			
PCC 145	✓								✓			
PCC 146		✓							✓			
PCC 147	✓	✓							✓			
PCC 148			✓						✓			
PCC 149	✓		✓						✓			
PCC 150		✓	✓						✓			
PCC 151	✓	✓	✓						✓			
PCC 152				✓					✓			
PCC 153	✓			✓					✓			
PCC 154		✓		✓					✓			
PCC 155	✓	✓		✓					✓			
PCC 156			✓	✓					✓			
PCC 157	✓		✓	✓					✓			
PCC 158		✓	✓	✓					✓			
PCC 159	✓	✓	✓	✓					✓			
PCC 160								✓	✓			
PCC 161		✓						✓	✓			
PCC 162			✓					✓	✓			
PCC 163		✓	✓					✓	✓			
PCC 164				✓				✓	✓			
PCC 165		✓		✓				✓	✓			
PCC 166			✓	✓				✓	✓			
PCC 167		✓	✓	✓				✓	✓			

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
Asphalt overlay of fractured slab:												
PCC 168										✓		
Bonded overlay combinations:												
PCC 169											✓	
PCC 170	✓										✓	
PCC 171		✓									✓	
PCC 172	✓	✓									✓	
PCC 173			✓								✓	
PCC 174	✓		✓								✓	
PCC 175		✓	✓								✓	
PCC 176	✓	✓	✓								✓	
PCC 177				✓							✓	
PCC 178	✓			✓							✓	
PCC 179		✓		✓							✓	
PCC 180	✓	✓		✓							✓	
PCC 181			✓	✓							✓	
PCC 182	✓		✓	✓							✓	
PCC 183		✓	✓	✓							✓	
PCC 184	✓	✓	✓	✓							✓	
PCC 185								✓			✓	
PCC 186		✓						✓			✓	
PCC 187			✓					✓			✓	
PCC 188		✓	✓					✓			✓	
PCC 189				✓				✓			✓	

Table 10 (continued). Feasible combinations of techniques for rehabilitation of concrete pavement.

№	Full-depth concrete repair / slab replacement	Partial-depth concrete repair	Undersealing / slabjacking	Load transfer restoration	Joint resealing	Grinding	Grooving	Pressure relief joints	Asphalt overlay	Asphalt or concrete overlay of fractured slab	Bonded concrete overlay	Unbonded concrete overlay
PCC 190		✓		✓				✓			✓	
PCC 191			✓	✓				✓			✓	
PCC 192		✓	✓	✓				✓			✓	
Unbonded overlay combinations:												
PCC 193												✓
PCC 194	✓											✓

Formation of Feasible Rehabilitation Strategies for Asphalt-Overlaid Concrete Pavements

1. Correct structural deficiency, if present.

If a structural deficiency exists, each of the rehabilitation strategy alternatives should include one of the following techniques:

- Asphalt overlay
- Asphalt or concrete overlay of fractured slab
- Unbonded concrete overlay
- Reconstruction

2. Correct functional deficiency, if present and not addressed by a structural improvement.

If a functional deficiency exists and is not addressed by a structural improvement, each of the rehabilitation strategy alternatives should include one of the following techniques.

- Cold milling
- Hot surface recycling
- Thin asphalt overlay
- Ultrathin concrete overlay

3. Select additional repair techniques to correct distresses not corrected by a structural or functional improvement.

The feasible combinations of techniques which may be used in nonoverlay or overlay rehabilitation of asphalt-overlaid concrete pavement are shown in Table 11. For example, rehabilitation strategy alternatives involving asphalt overlay may be formed using some or all of the combinations listed as COMP 12 through COMP 23. Note that reconstruction is not shown in the table because it does not require combination with any other repair or resurfacing techniques.

4. Select a drainage improvement to correct drainage deficiency, if present.

If a drainage deficiency exists, a drainage improvement option may be considered for inclusion in some or all of the rehabilitation strategy alternatives. Drainage improvement options for in-service pavements may include retrofitting or replacing longitudinal subdrains and outlets, or daylighting the base by replacing shoulder base material and repaving the shoulders.

These drainage improvement options are presented with the caveat that it is difficult to predict what effects drainage improvement efforts will have on the performance of the rehabilitated pavement. Reconstruction alternatives may of course be developed to include design of a completely new drainage system.

Table 11. Feasible combinations of techniques for rehabilitation of asphalt-overlaid concrete pavement.

№	Full-depth concrete or composite repair	Partial-depth asphalt repair	Cold milling	In-place recycling	Asphalt overlay	Asphalt or concrete overlay Of fractured slab	Unbonded concrete overlay
Nonoverlay combinations:							
COMP 1	✓						
COMP 2		✓					
COMP 3	✓	✓					
COMP 4			✓				
COMP 5	✓		✓				
COMP 6		✓	✓				
COMP 7	✓	✓	✓				
COMP 8				✓			
COMP 9	✓			✓			
COMP 10		✓		✓			
COMP 11	✓	✓		✓			
Asphalt overlay combinations:							
COMP 12					✓		
COMP 13	✓				✓		
COMP 14		✓			✓		
COMP 15	✓	✓			✓		
COMP 16			✓		✓		
COMP 17	✓		✓		✓		
COMP 18		✓	✓		✓		
COMP 19	✓	✓	✓		✓		
COMP 20				✓	✓		
COMP 21	✓			✓	✓		
COMP 22		✓		✓	✓		
COMP 23	✓	✓		✓	✓		

Table 11 (continued). Feasible combinations of techniques for rehabilitation of asphalt-overlaid concrete pavement.

№	Full-depth concrete or composite repair	Partial-depth asphalt repair	Cold milling	In-place recycling	Asphalt overlay	Asphalt or concrete overlay Of fractured slab	Unbonded concrete overlay
Asphalt overlay of fractured slab:							
COMP 24						✓	
COMP 25			✓			✓	
COMP 26				✓		✓	
Unbonded concrete overlay combinations:							
COMP 27							✓
COMP 28	✓						✓

Rehabilitation Design

Reconstruction Design

The thickness design, asphalt or concrete mix design, and in the case of reconstruction in concrete, joint and reinforcement design, are essentially the same as for new pavement design. The only additional design consideration may be modifications to the asphalt or concrete mix design when recycled materials are to be used.

Overlay Design

The thickness and the asphalt concrete mix must be designed for asphalt overlays of all pavement types. The thickness, concrete mix, joint details, and reinforcement details must be designed for concrete overlays of all pavement types.

The two most commonly used approaches to structural design of asphalt overlays of asphalt pavements are (1) the structural deficiency approach, exemplified by the 1993 AASHTO[1] procedure; and (2) the deflection-based approach, exemplified by the Asphalt Institute[4] procedure. Less common is the mechanistic approach, in which fatigue, rutting, and sometimes thermal cracking are predicted using mechanistic-empirical models. Among the few State DOTs that have developed a mechanistic-empirical design procedure for asphalt overlays of asphalt pavements are Washington[61] and Nevada.[62,63] Design of asphalt overlays of asphalt pavements is described in more detail in Appendix B of this Guide.

The most commonly used approach to structural design of asphalt overlays of concrete pavements and asphalt-overlaid concrete pavements is the structural deficiency approach, exemplified by the 1993 AASHTO procedure.[1] Design of asphalt overlays of concrete and asphalt-overlaid concrete pavements is described in more detail in Appendix B.

Structural design for an asphalt or concrete overlay of a cracked and seated or broken and seated concrete may be done using either flexible pavement or rigid pavement overlay design methods. Structural design for an asphalt or concrete overlay of a rubblized pavement is similar to that for a new asphalt or concrete pavement on a high-strength granular base. Design of asphalt overlays of fractured concrete pavements is described in more detail in Appendix B of this Guide.

The structural design of a conventional concrete overlay of an asphalt pavement is comparable to that of new concrete pavements or unbonded concrete overlays. Ultrathin concrete overlays require special considerations in their thickness and joint design. Design of concrete overlays of asphalt pavements is described in more detail in Appendix B of this Guide.

The most commonly used approach to thickness design for bonded concrete overlays of concrete pavements and unbonded concrete overlays of concrete and asphalt-overlaid concrete pavements is the Corps of Engineers method.[64,65] This structural deficiency approach is also used in the 1993 AASHTO Guide.[1] An alternative approach to design of an unbonded overlay is to design it as if it were a new pavement on a rigid base. Design of bonded and unbonded concrete overlays is described in more detail in Appendix B of this Guide.

Load Transfer Design

The dowel load transfer system must be designed for full-depth repairs in jointed concrete pavements and for load-transfer restoration in jointed concrete pavements. This involves selecting an appropriate dowel diameter and layout pattern.

Subdrainage Design

Subdrainage improvement design may involve design of the retrofit longitudinal subdrains and outlets, or gradation selection for open-graded granular material placed along the shoulder to daylight the existing base. Subdrainage design is described in more detail in Reference 66.

Materials Selection

Rehabilitation materials selection is an activity conducted in conjunction with rehabilitation design. Any conventional or proprietary overlay materials, patching materials, joint and crack sealing materials, bonding materials, backfilling materials, subsealing materials, subdrainage materials, etc. which differ by rehabilitation strategy alternative must be identified. These materials must be identified so their costs may be determined. The materials selected may also influence the predicted rehabilitation performance.

Rehabilitation Performance Prediction

Rehabilitation performance prediction involves, as a minimum, predicting the time (either in years or accumulated axle loadings) at which each rehabilitation strategy alternative will reach a level of condition requiring follow-up rehabilitation. Rehabilitation performance prediction may also involve predicting the shape of the performance curve over this life.

The performance of a given rehabilitation strategy is very difficult to predict. Rehabilitation performance is particularly sensitive to the following factors:

- Appropriateness of use of the technique for the type of distress present, and for the point in the structural life of the pavement at which the rehabilitation is applied;
- The combination of techniques applied;
- For overlay and reconstruction techniques, the structural design (thickness, joints, etc.)
- The extent, type, and construction quality of preoverlay repairs;
- The materials used;
- The construction quality of the rehabilitation; and
- The truck traffic level.

The principal factors that influence, to different degrees depending on the rehabilitation treatment, the performance of that treatment, are discussed below.

Pavement Type

Several rehabilitation techniques are applicable only to one type of pavement, but several others are applicable to more than one pavement type. The same rehabilitation technique may exhibit different performance when applied to different pavement types. For example, a given thickness of asphalt overlay will manifest different types of distress, and possibly different performance lives, when applied to an asphalt pavement, a jointed concrete pavement, and a continuously reinforced concrete pavement.

Pavement Condition

Is the treatment appropriate at this point in the pavement´s life, given the types and severities of distress present? For example, surficial improvements to an aged asphalt pavement surface are not appropriate if the pavement exhibits such substantial structural distress that it is clearly in need of resurfacing or replacement sometime soon. Some assessments about appropriateness of rehabilitation treatment timing can be made on the basis of judgment, but usually an analysis of predicted performance and costs is needed to assess whether or not a treatment is cost-effective at a given point in time.

Concurrent Work

Is the treatment compatible with other treatments being considered as part of the rehabilitation effort? Some treatments supercede the need for other treatments. Some treatments affect not only the traffic lane being treated but also adjacent traffic lanes and/or shoulders.

Design Traffic

Many rehabilitation techniques require some design for the traffic anticipated over the performance period of the rehabilitation strategy. Structural overlay thicknesses, as well as load transfer systems in full-depth repairs, load transfer restoration, and jointed concrete overlays, must be designed for the volume of truck axle loadings anticipated. Asphalt mixes must be designed not only for the traffic volumes but also the the tire pressures anticipated.

Adequacy of Design Procedures and Performance Prediction Methods

Whether or not not a rehabilitation technique exhibits the performance expected by an agency is also related to whether or not the rehabilitation design procedures and rehabilitation performance prediction methods used by the agency for the technique are adequate and realistic. The rehabilitation design and performance models used may fail to take into account correctly one or more important factors which will influence the performance of the technique.

Rehabilitation performance models that have poor predictive capability for the local conditions may yield insufficiently reliable rehabilitation performance predictions. An example would be an asphalt overlay rutting model that is applicable to the asphalt concrete mix design parameters of some other agency and/or the climatic conditions of some other region.

Another concern with rehabilitation design and performance models is that they may (a) fail to consider some factors which are important to the performance of the technique, and/or (b) underestimate or overestimate the sensitivity of rehabilitation performance to factors which are considered.

Adequacy of Characterization of Existing Pavement Condition

The appropriate selection of some rehabilitation techniques and their adequate design depends on how adequately the condition of the existing pavement condition is characterized. For example, a structural overlay may be designed to satisfy a structural deficiency in an existing pavement, but this structural deficiency may be underestimated or overestimated if it is based on very limited condition data.

As-Designed versus As-Placed Characteristics

Many rehabilitation design and performance models include parameters related to as-placed characteristics of the materials used: asphalt mix stability, density, gradation, granular layer gradation and density, concrete repair material strength, etc. The performance of many rehabilitation techniques depends on how well the as-placed characteristics of the materials agree with those assumed in the design and performance prediction of the rehabilitation.

Traffic Restrictions

Whether certain rehabilitation techniques can be used, and the care with which they can be constructed, sometimes depends on the amount of time that disruptions to traffic are permitted for the rehabilitation work. On the other hand, some techniques which may require longer initially to construct may be preferable because they will require fewer traffic interruptions for maintenance in the future. The performance of techniques which are more sensitive to the diligence of follow-up maintenance may be diminished if traffic restrictions make this follow-up maintenance infeasible. Some assessments about how traffic restrictions may affect rehabilitation performance can be made on the basis of judgment, but usually an analysis of the costs related to traffic flow disruption is needed to assess whether rehabilitation strategies under consideration differ significantly in performance or cost-effectiveness because of traffic restrictions.

Climate

Asphalt mixes, jointed concrete overlay joint spacings, and subdrainage system details are among the rehabilitation aspects which must be designed for the climate in which the rehabilitation is to be applied.

Past Experience with the Technique

An agency's past experience with a rehabilitation technique is reflected in many aspects: in its appropriate selection for a given pavement, in the design procedure and construction specifications the agency uses for the technique, in the experience of the agency's personnel in supervsing the construction of the technique, and the experience of available contractors in constructing the technique. All of these aspects can influence the performance achieved with the technique.

Pretreatment Preparation

The performance of some rehabilitation treatments depends greatly on the type, extent, and quality of repairs done prior to application of the treatment. For example, the performance of an asphalt overlay or a bonded concrete overlay often depends on amount of existing distress repaired, the types of repairs done, the construction quality of the repairs, and the amount of distress left unrepaired.

Materials

Material selection is particularly crucial to the performance of some rehabilitation treatments, such as asphalt patching, and concrete full-depth and partial-depth repair.

Equipment

For some rehabilitation treatments, the specific types of equipment used in construction influence the performance of the treatment. For example, gang drilling machines that can simultaneously drill all of the dowel holes for one side of a full-depth repair provide better dowel alignment than individually drilled holes.[67]

Construction Procedures

Among the aspects of the rehabilitation construction procedures that influence, to varying degrees, the performance of many rehabilitation techniques are the sequencing of activities involved in a rehabilitation effort, the specific construction techniques employed, the control of construction quality, and the appropriateness of the construction procedures used for the ambient conditions during construction.

Typical Ranges of Rehabilitation Service Life

Typical ranges of service life are given in Table 12 and discussed below, first for reconstruction in asphalt and in concrete, and then for each of the other major rehabilitation techniques described in this Guide for each pavement type. These ranges are general estimates only, expressed in years, not including consideration of truck traffic level. The ranges are intended to represent the "conventional wisdom" about the service lives that may reasonably be expected of the different rehabilitation techniques. Service life estimates used by specific State DOTs or other agencies are cited where possible for comparison.

Typical Ranges of Service Lives for Reconstruction

- **Reconstruction in asphalt: 15 – 25 years.** Ohio estimates that an asphalt pavement will require an overlay in year 10 to 15, and another overlay, with patching and milling, in year 18 to 25.68 Mississippi estimates that an asphalt pavement will require an overlay in years 10 and 20.69 Pennsylvania estimates that an asphalt pavement will require patching plus cold milling and replacement of the top 1.5 to 2 inches of asphalt concrete in year 10, and patching and overlay in years 20 and 30.70 New York estimates a service life of 15 years for reconstruction in asphalt, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77] Utah estimates that asphalt pavements with ADT of 5000 or more will require structural overlays

Table 12. Typical ranges of service lives for rehabilitation treatments

Treatment	Typical range of service life, years
Reconstruction:	
Reconstruction in asphalt	15 – 25
Reconstruction in concrete	20 – 30
Asphalt pavement rehabilitation:	
Structural asphalt overlay of asphalt pavement	8 – 15
Structural concrete overlay of asphalt pavement	20 – 30
Surface recycling without overlay	4 – 8
Nonstructural asphalt overlay of asphalt pavement	4 – 8
Nonstructural (ultrathin) concrete overlay of asphalt pavement	5 – 15
Asphalt patching without overlay	4 – 8
Concrete pavement rehabilitation:	
Structural asphalt overlay of concrete pavement	8 – 15
Asphalt or concrete overlay of fractured concrete slab	15 – 25
Unbonded concrete overlay of concrete pavement	20 – 30
Nonstructural asphalt overlay of concrete pavement	4 – 8
Bonded concrete overlay of concrete pavement	15 – 25
Restoration without overlay	5 – 15
Asphalt-overlaid concrete pavement rehabilitation:	
Structural asphalt overlay of AC/PCC pavement	8 – 15
Asphalt or concrete overlay of fractured concrete slab	15 – 25
Unbonded concrete overlay of AC/PCC pavement	20 – 30
Surface recycling without overlay	4 – 8
Nonstructural asphalt overlay of AC/PCC pavement	4 – 8
Nonstructural (ultrathin) concrete overlay of AC/PCC pavement	5 – 15

in years 15 and 30, and that asphalt pavements with ADT less than 5000 will require a structural overlay in year 23.[74] Wisconsin estimates a service life of 18 years for reconstruction in asphalt.[71] Indiana estimates a service life of 20 years for reconstruction in full-depth asphalt.[72] Arizona estimates a service life of 15 years for asphalt pavements.[73]

- **Reconstruction in concrete: 20 – 30 years.** Ohio estimates that a concrete pavement will require rehabilitation (full-depth and/or partial-depth repairs, and diamond grinding or an overlay in year 18 to 25.[68] New York estimates a service life of 30 years for reconstruction in concrete, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77] Pennsylvania estimates that a concrete pavement will require restoration in year 20, an overlay in year 30, and subsequent overlays at intervals of 8 years.[70] Utah estimates that concrete pavements of all functional classes will require restoration every 10 years.[74] Indiana estimates a service life of 30 years for reconstruction in concrete.[72] Arizona estimates a service life of 20 years for concrete pavements.[73]

Typical Ranges of Service Lives for Asphalt Pavement Rehabilitation Strategies

- **Asphalt overlay: 8 – 15 years.** New York estimates a service life of 15 years for 3 or more inches of new asphalt without cold milling or hot surface recycling, and for 3 or more inches of new asphalt after cold milling to a depth of 1.5 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks. [77] Vermont estimates a service life of 6 to 12 years for 2 to 5 inches of asphalt overlay.[75] West Virginia estimates a service life of 8 years for asphalt overlays of asphalt pavements.[76] Wisconsin estimates a service life of 10 to 14 years for asphalt overlays of asphalt pavements.[71] Arizona estimates a service life of 10 years for asphalt overlays of asphalt pavements.[73]
- **Conventional concrete overlay: 20 – 30 years.** The typical range of service life for conventional concrete overlay is the same as for reconstruction in concrete.
- **Hot surface recycling without overlay: 4 – 8 years.** New York estimates a service life of 8 years for hot in-place recycling of 1 to 1.5 inches, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77] Vermont estimates a service life of 6 to 10 years for hot in-place recycling of 1 to 3 inches.[75]
- **Thin asphalt overlay: 4 – 8 years.** New York estimates a service life of 8 years for 1- to 1.5-inch asphalt overlays of asphalt pavements, and also for 1- to 1.5 inch replacement of asphalt concrete removed by cold milling. New York estimates a service life of 15 years for 1.5 inches of new asphalt after hot in-place recycling to a depth of 1.5 inches, or cold in-place recycling to a depth of 3 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks, except the option involving cold in-place recycling, which is only considered suitable for pavements with less than 4000 ADT per lane.[77] Vermont estimates a service life of 5 to 8 years for thin asphalt overlays.[75] Wisconsin estimates a service life of 6 to 9 years for thin asphalt overlays of asphalt

pavements.[71] Indiana estimates a service life of 5 to 8 years for thin asphalt overlay of asphalt, with millling.[72]

- **Ultrathin concrete overlay: 5 – 15 years.** This is merely a tentative estimate, as there are no service life estimates to be found in the literature for this technique. The American Concrete Pavement Association has a design procedure for ultrathin whitetopping.

- **Patching without overlay: 4 – 8 years.** The American Concrete Pavement Association estimates a service life of 5 to 7 years for asphalt patching without overlay.[57]

Typical Ranges of Service Lives for Concrete Pavement Rehabilitation Strategies

- **Asphalt overlay: 8 – 15 years.** The American Concrete Pavement Association estimates service lives of 8 to 12 years, 10 to 15 years, and 10 to 20 years, depending on design, for asphalt overlays of Interstate, primary/secondary, and municipal concrete pavements respectively.[57] Ohio estimates that an asphalt overlay of a concrete pavement will require rehabilitation (full-depth rigid repairs, milling, and a follow-up overlay) every 8 to 12 years.[68] New York estimates a service life of 15 years for 3- to 4-inch asphalt overlays of concrete pavements, and a service life of 8 years for sawed and sealed joints in asphalt overlays, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77] West Virginia estimates a service life of 8 years for first asphalt overlays of concrete pavements.[76] Indiana estimates a service life of 10 years for asphalt overlay of jointed concrete pavement without sawing and sealing joints, or 13 years with sawing and sealing joints. Indiana estimates a service life of 12 to 15 years for asphalt overlay of continuously reinforced concrete pavement.[72] Arizona estimates a service life of 10 years for asphalt overlay of concrete pavement.[73]

- **Asphalt overlay of fractured concrete slab: 15 – 25 years.** Ohio estimates that an asphalt overlay of a fractured concrete slab will require a thin overlay (1.25 to 4 inches) in year 8 to 12, and a thick overlay (4 to 8 inches), along with patching and milling, in year 16 to 22.[68] New York estimates a service life of 15 years for either a 5-inch asphalt overlay of a cracked and seated concrete pavement, or a 6-inch asphalt overlay of a rubblized concrete pavement, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77] West Virginia estimates a service life of 8 years for asphalt overlay of broken and seated or rubblized concrete pavement.[76] Wisconsin estimates a service life of 15 to 18 years for an asphalt overlay of a rubblized concrete pavement.[71] Indiana estimates a service life of 12 to 15 years for an asphalt overlay of a cracked and seated concrete pavement, and a service life of 20 years for an asphalt overlay of a rubblized concrete pavement.[72]

- **Unbonded concrete overlay: 20 – 30 years.** The typical range of service life for unbonded concrete overlay is the same as for reconstruction in concrete. The American Concrete Pavement Association estimates a service life of 30 or more years, depending on design, for an unbonded concrete overlay.[57] West Virginia estimates a service life of 20 years for unbonded concrete overlay.[76]

- **Thin asphalt overlay: 4 – 8 years.**

- **Bonded concrete overlay: 15 – 25 years.** The American Concrete Pavement Association estimates a service life of 15 to 25 years, depending on design, for a bonded concrete overlay.[57] New York estimates a service life of 20 years, with joint and crack resealing be required at 8-year intervals for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77]

- **Restoration: 5 – 15 years.** Overall service life depends on the combination of techniques used, and the extent, materials, and construction quality of each. The American Concrete Pavement Association estimates the following service lives for different individual restoration techniques: [57]

    - Joint resealing: 5 – 15 years
    - Partial-depth spall repair: 10 – 15 years
    - Diamond grinding: 10 – 15 years
    - Full-depth repair: 10 – 15 years
    - Load transfer restoration: 8 – 10 years

  New York estimates the following service lifes for certain restoration techniques and combinations:[77]

    - Joint resealing alone: 2 years
    - Joint resealing when in conjunction with partial-depth spall repair: 8 years
    - Partial-depth spall repair: 10 years
    - Diamond grinding: 5 years
    - Slab replacement: lasts as long as existing pavement

  These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks.

Utah estimates the following service lives for certain restoration techniques and combinations:[74]

- Joint resealing when in conjunction with partial-depth spall repair: 10 years
- Partial-depth spall repair: 10 years
- Diamond grinding in conjunction with load transfer restoration and undersealing and/or slabjacking: 10 years

Wisconsin estimates a service life of 5 to 10 years for diamond grinding done in conjunction with other repairs.[71] Indiana estimates a service life of 5 to 15 years for concrete pavement restoration.[72] Arizona estimates a service life of 14 years for diamond grinding plus joint resealing, and a service life of 10 years for grooving plus joint resealing.[73]

A 1998 study conducted for the ACPA found that diamond-ground pavements last an average of 13.5 years or 12 million ESALs before regrinding, overlay, or reconstruction.[78]

Typical Ranges of Service Lives for Asphalt-Overlaid Concrete Pavement Rehabilitation Strategies

- **Asphalt overlay: 8 – 15 years.** New York estimates a service life of 15 years for 3 or more inches of new asphalt without cold milling or hot surface recycling, and for 3 or more inches of new asphalt after cold milling to a depth of 1.5 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks. [77] West Virginia estimates a service life of 8 years for the second asphalt overlay of existing asphalt-overlaid concrete pavement.[76] Wisconsin estimates a service life of 4 to 8 years for thin asphalt overlays of existing asphalt-overlaid concrete pavements.[71]

- **Asphalt overlay of fractured concrete slab: 15 – 25 years.** Ohio estimates that an asphalt overlay of a fractured concrete slab will require a thin overlay (1.25 to 4 inches) in year 8 to 12, and a thick overlay (4 to 8 inches), along with patching and milling, in year 16 to 22.[68] New York estimates a service life of 15 years for either a 5-inch asphalt overlay of a cracked and seated concrete pavement, or a 6-inch asphalt overlay of a rubblized concrete pavement, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77]

- **Unbonded concrete overlay: 20 – 30 years.** The typical range of service life for unbonded concrete overlay is the same as for reconstruction in concrete. The American Concrete Pavement Association estimates a service life of 30 or more years, depending on design, for an unbonded concrete overlay.[57]

- **Hot surface recycling without overlay: 4 – 8 years.** New York estimates a service life of 8 years for 1- to 1.5-inch hot surface recycling, for highways with ADT between 12,000 and 35,000, and about 5 percent trucks.[77]

- **Thin asphalt overlay: 4 – 8 years.** New York estimates a service life of 8 years for 1- to 1.5-inch asphalt overlays of existing asphalt-overlaid concrete pavements, and also for 1- to 1.5 inch replacement of asphalt concrete removed by cold milling. The same agency estimates a service life of 15 years for 1.5 inches of new asphalt after hot in-place recycling to a depth of 1.5 inches, or cold in-place recycling to a depth of 3 inches. These estimates apply to highways with ADT between 12,000 and 35,000, and about 5 percent trucks, except the option involving cold in-place recycling, which is only considered suitable for pavements with less than 4000 ADT per lane.[77]

- **Ultrathin concrete overlay: 5 – 15 years.** This is merely a tentative estimate, as there are no service life estimates to be found in the literature for this technique. The American Concrete Pavement Association has a design procedure for ultrathin whitetopping.

Comparison of Predicted Performance with Performance Standards

The predicted performance of each rehabilitation strategy should be compared with the performance standards established by the agency. These may include roughness and/or condition criteria. Any strategy whose performance is not predicted to be acceptable over the analysis period must be modified. The modifications needed to achieve acceptable predicted performance may include one or more of the following:

- Assignment of follow-up rehabilitation,
- Modification of repair quantities,
- Modification of the rehabilitation design,
- Selection of different materials.

The designer is advised to give careful consideration to the measure or measures of pavement performance by which the alternatives are judged. Pavement condition may be expressed in several ways: serviceability (ride quality), measured roughness, friction, quantity and severity of key distresses, a condition index which has been developed to collectively quantify distress and/or ride quality, etc. The two questions to consider in selecting an appropriate performance measure by which to compare alternatives are:

- What performance measure or measures are the best indicators of (a) the remaining life of the pavement structure, and (b) the cost of repairing or replacing the pavement structure?
- What performance measure or measures lend themselves best to a common basis of comparison for all of the alternatives under consideration?

Construction and Traffic Control Plan

Development of a construction and traffic control plan for each rehabilitation strategy alternative involves the following:

- Establishing the sequencing of construction rehabilitation techniques.
- Estimating the duration of each step in the rehabilitation construction.
- Identifying other activities which must be included in the construction plan (e.g., construction of crossovers, repositioning guardrails and signs, raising or reconstructing under bridges, etc.).
- Developing a plan for traffic control during each phase of construction.

One State DOT's estimated production rates for paving and rehabilitation activities are given in Table 13. Production rates such as these may be used to estimate lane closure durations and costs. These particular amounts are shown only for purposes of illustration. It is preferable to obtain estimates from contractors familiar with the particular rehabilitation project in question.

Table 13. Example paving and rehabilitation work production rates.[68]

Work Item	Production Rate	Notes
Wearing course removal	11,250 sy/day	
Pavement removal	2250 sy/day	A
Base removal	1000 cy/day	
Excavation not including embankment	2500 cy/day	
Subgrade compaction	1 day/lane	
Proof rolling	48,750 sy/day	
Lime stabilizing subgrade soil	2125 sy/day	
Cold milling of asphalt	8750 sy/day	B
Cold milling of concrete	8750 sy/day	C
Partial-depth concrete repair	1625 sy/day	D
Full-depth concrete repair	875 sy/day	D
Pavement sawing	1 day/lane	
Bituminous aggregate base	875 cy/day	
Aggregate base	1250 cy/day	
Concrete base	2875 sy/day	D
Cement-treated free-draining base	2875 sy/day	D
Asphalt-treated free-draining base	3125 sy/day	
Unstabilized drainage base	3750 sy/day	
Tack coat	neglect	
Bituminous prime coat	neglect	
Seal coat	neglect	
Sawing and sealing	1875 lin ft/day	
Asphalt concrete surface course	1124 cy/day	C, E
Asphalt concrete intermediate course	625 cy/day	
Concrete pavement mainline	4750 sy/day	D
Concrete pavement shoulders	3175 sy/day	D
Continuously reinforced concrete pavement	1875 sy/day	
Cracking and seating	12,500 sy/day	
Rubblize and roll	2500 sy/day	
Joint clean/seal, all types	13750 lin ft/day + 1 day/lane	C

Notes:

A – For situations where shoulders are being removed for replacement, pavement removal and wearing course removal can be done simultaneously.

B – Where conditions permit the pavement to be opened to traffic at the end of each work day, the production rate for this item should be doubled. When the dropoff between lanes is too large to permit leaving the pavement open to traffic, the given production rate should be used without doubling.

C – Production rates for these items have been adjusted to reflect the fact that the pavement is open to traffic during the part of the day when the work is not being performed.

D – All concrete pavement work items do not include curing time. The curing time should be added to the total number of days of lane closure, where applicable.

E – Where sawing and sealing is specified, use 1 day/lane for AC surface course.

Chapter 6

Guidelines for Life-Cycle Cost Analysis of Pavement Rehabilitation Strategies

The objective of life-cycle cost analysis is to evaluate the economic effectiveness of different mutually exclusive investment alternatives over a certain time period and to identify the most cost-effective alternative. The selection of an appropriate rehabilitation strategy for a pavement should consider all of the costs and benefits that will be incurred as a result of the selection of that strategy. These costs and benefits should be estimated over a time frame that is sufficiently long to reflect differences in performance among different strategy alternatives. This period of time is generally referred to as the **analysis period**. A fair comparison among alternatives over the analysis period requires that their associated costs be expressed in terms of some common monetary measure. The calculation and comparison of the costs and benefits of different alternatives over the analysis period is called **life-cycle cost analysis**.

The period of time for which either a new pavement or a rehabilitation treatment is designed to serve is often called the **design period**. In the context of rehabilitation strategy selection, it may be more convenient to use the term **performance period**. Some rehabilitation treatments, such as overlays, can be designed for a specific time period or number of traffic loadings, but some others, such as diamond grinding or subdrainage retrofitting, cannot. For many rehabilitation techniques, the best estimate of the life of the technique must come from field performance observations or empirical models developed from field performance data. Thus, the term performance period encompasses more generally the expected life of any rehabilitation treatment, whether or not it is designed.

A life-cycle cost analysis of pavement rehabilitation strategy alternatives, when done correctly, permits the identification of the strategy which yields the best value, by providing the desired performance at the lowest cost over the analysis period. Ideally, a comprehensive life-cycle cost analysis of pavement rehabilitation strategy alternatives would consider quantitatively all of the costs to be incurred by both the agency and the users over the analysis period. However, some of these costs are difficult to quantify, necessitating some simplifications to the life-cycle cost analysis. Furthermore, differences of opinion exist about how to quantify some of the inputs to the life-cycle cost analysis process. Each agency must make its own choices about the level of complexity it desires in the life-cycle cost analysis it conducts, and about the way it defines the inputs to the analysis.

Select Analysis Period

The analysis period is the time over which alternatives are compared. For comparison of rehabilitation strategy alternatives, the analysis period should begin at the end of the performance period of the original pavement, as illustrated in Figure 20.

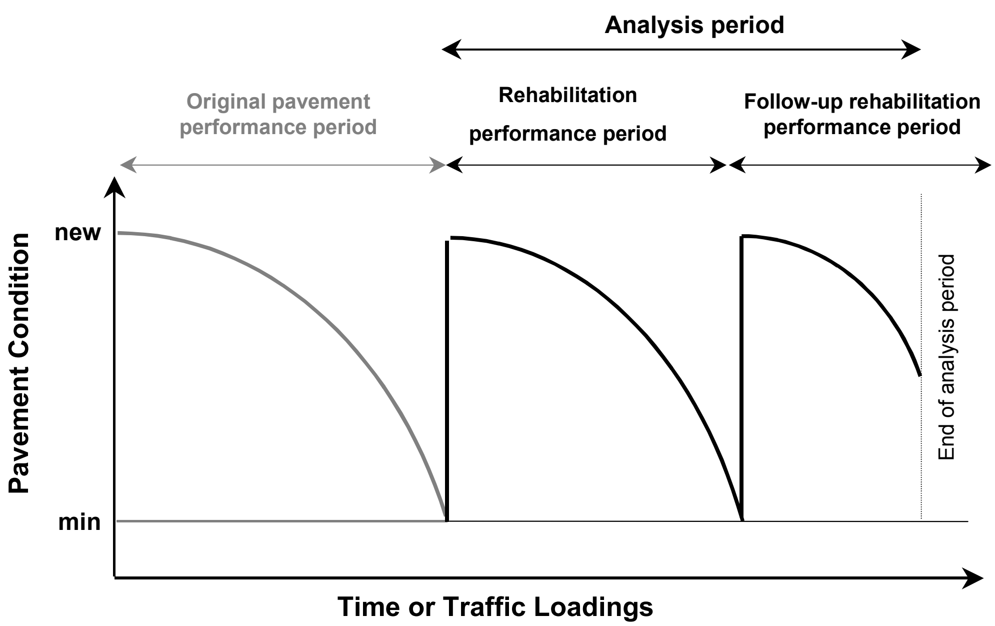

Figure 20. Rehabilitation strategy analysis period beginning at end of original pavement performance period.

When all of the investment alternatives have the same performance period, the most common way of defining the analysis period is to use the common performance period. This situation is illustrated in the following example.

Analysis Period Example 1: common performance period, different performance.

Figure 21 illustrates the performance curves for three rehabilitation alternatives which are designed for a common performance period, but are expected to exhibit notably different performance over this performance period. (Note that in Figure 21, all three rehabilitation alternatives are shown to achieve the same initial improvement in pavement condition, although this may not necessarily be true in all cases.) It is reasonable to compare these alternatives over an analysis period equal to the performance period. A longer analysis period could also be used, in which case, follow-up rehabilitation treatments would have to be assigned to all three alternatives.

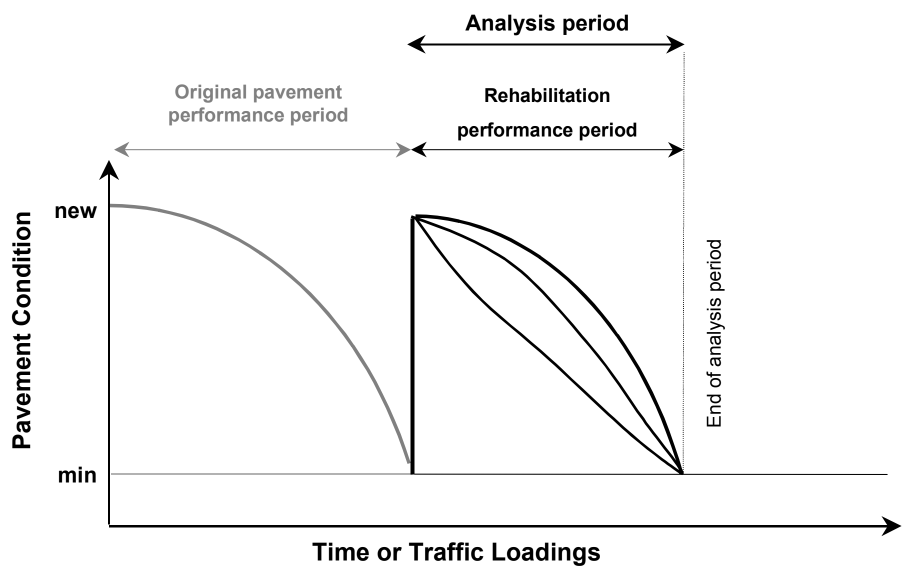

Figure 21. Selection of analysis period for alternatives with common performance period, but notably different performance (Example 1).

The following options are available for defining the analysis period in the analysis of investment alternatives with different performance periods:

- The least common multiple of the performance periods of all of the alternatives.
- The shortest of the performance periods among the alternatives.
- The longest of the performance periods among the alternatives.
- Some other time period.

When considering relatively long performance periods, such as is the case in pavement rehabilitation strategy selection, the use of the least common multiple of the performance periods can result in an extremely long and unrealistic analysis period. The use of the shortest performance period as the analysis period can adversely affect those alternatives with better long-term performance, and favor those with short performance periods. Among the options listed above, the use of the longest of the performance periods as the analysis period is recommended as the most appropriate for pavement rehabilitation strategy selection.

Analysis Period Example 2: unequal performance periods.

Figure 22 illustrates the performance curves for three rehabilitation alternatives which are expected to have different performance periods, that is, they are expected to be able to keep the pavement condition above the minimum acceptable level for different lengths of time. The analysis period is recommended to be no less than the performance period of the longest-surviving alternative. Using a shorter analysis period (e.g., equal to the performance period of one of the shorter-lived alternatives) would not fully capture the anticipated differences in performance.

In this example, selecting an analysis period equal to the performance period of the longest-surviving alternative requires that follow-up rehabilitation treatments be assigned to the other alternatives in order to fill out the analysis period. A longer analysis period could also be used, in which case, follow-up rehabilitation treatments would have to be assigned to all three alternatives. This is illustrated in Figure 23.

As illustrated in both Figures 22 and 23, one or more of the alternatives may have a follow-up rehabilitation performance period which extends beyond the end of the analysis period. This implies that, if that alternative were chosen, the pavement structure would have some remaining useful life before its condition would again fall to the minimum acceptable level. When the different alternatives considered have different remaining lives at the end of the analysis period, this difference should be taken into account by assigning some residual or salvage value to the alternative. This topic is discussed more later.

The analysis period may be defined using some other time period. For example, the Federal Highway Administration´s policy statement on life-cycle cost analyis recommends an analysis period of at least 35 years for all pavement projects, i.e., new construction as well as rehabilitation.[79] However, it may be difficult in some instances to use so long an analysis period when comparing rehabilitation strategy alternatives.

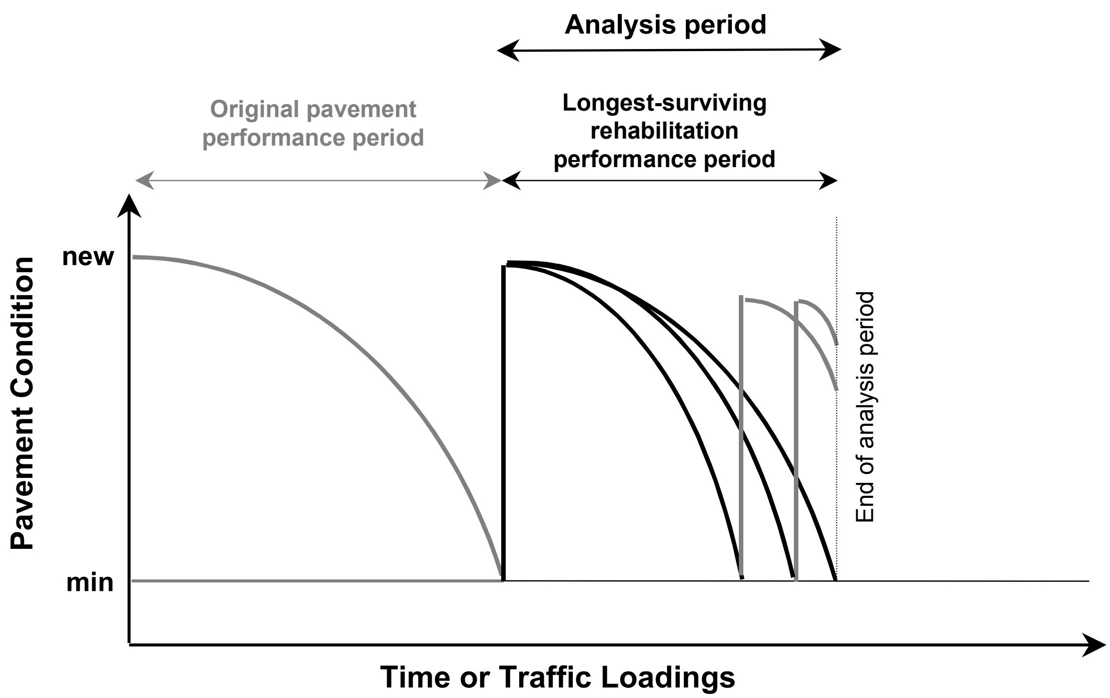

Figure 22. Selection of analysis period for alternatives with unequal performance periods (Example 2).

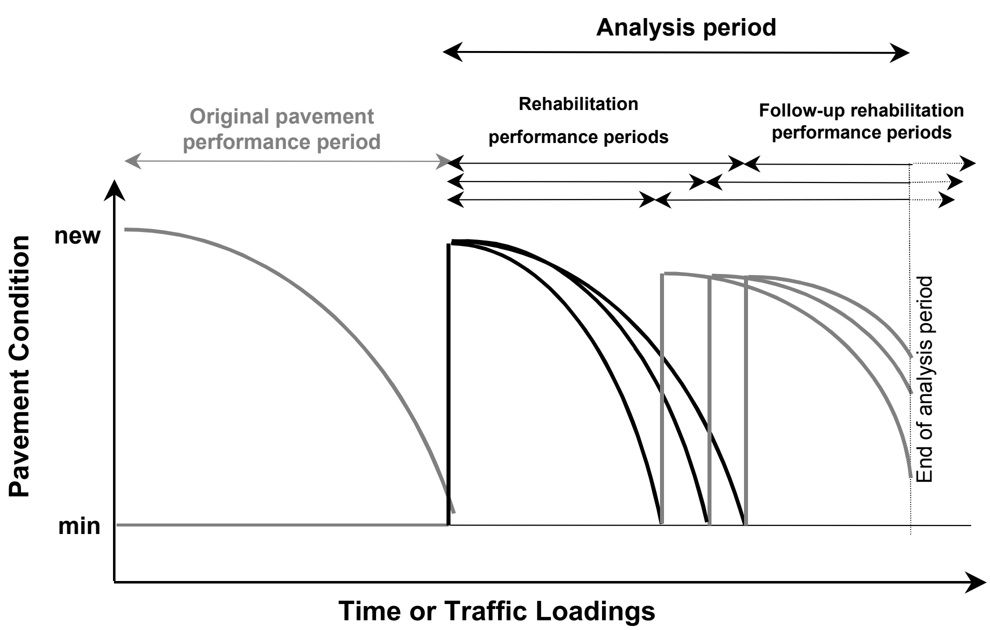

Figure 23. Selection of analysis period to encompass follow-up rehabilitation for all alternatives (Example 2).

Consider, for example, a situation in which alternatives such as reconstruction or major structural improvement are being considered. Long-lived strategies such as these can be designed for performance periods of 20 years or more. It is reasonable to presume that one could anticipate what one or at most two follow-up rehabilitation treatments might be applied over the next 15 years or so, and thereby fill out a 35-year analysis period.

Consider, however, another situation in which such long-lived strategies are not being considered (because of geometric restrictions, perhaps, or funding limitations). When comparing rehabilitation strategy alternatives that can only reasonably be expected to achieve performance periods of, say, 5 to 15 years, it is not reasonable to presume that one could anticipate what series of three or more rehabilitation treatments would be applied over a time frame as long as 35 years, and reliably predict the performance of each treatment. The major obstacle is the very limited state of knowledge about the performance of second and subsequent rehabilitation treatments.

What is most important about the selection of an appropriate analysis period is that it be sufficiently long to reflect significant differences in performance among the different strategy alternatives.

Select Discount Rate

The term **discount rate** is commonly used in engineering economics to refer to the rate of change over time in the true value of money, taking into account fluctuations in both investment interest rates and the rate of inflation. A discount rate is used to determine the present value of all initial and future costs, during the analysis period, associated with each alternative considered in the life-cycle cost analysis. When inflation is taken into consideration in the selection of the discount rate, all initial and future cost items should be expressed in constant dollars, i.e., in terms of the costs of those items if they were incurred in the year in which the life-cycle cost analysis were conducted.

The discount rate is approximately equal to the interest rate minus the inflation rate. Interest rates and inflation rates both fluctuate over time, but the difference between the two, while not constant, is more consistent. The discount rate selected should be as realistic as possible, taking into account past trends in interest and inflation rates over relatively long time periods, as well as future economic projections.

The appropriate interest rate to be used in economic analysis of public projects is a topic of much debate. The various arguments concerning the correct philosophy to adopt in the selection of an appropriate interest for a publicly funded project fall into the following groups.[80]

1. ***Philosophy 1: A zero interest rate is appropriate when tax monies are used for financing***. Advocates of a zero interest rate when tax monies (e.g., highway user taxes) are used argue that such funds are "free money" because no principal or interest payments are required. The counter argument is that zero or very low interest rates can produce positive benefit/cost ratios even for very marginal projects, and thereby take money away from more truly deserving projects. A zero interest rate also fails to discount future expenditures, making tomorrow's relatively uncertain expected costs just as important to the decision as today's known costs.

2. ***Philosophy 2: The interest rate need only reflect the "societal rate of time preference,"*** that is, the interest rate that "reflects the government's judgement about the relative value which the community as a whole assigns, or which the government feels it ought to assign, to present versus future consumption." [81] The societal time preference rate "need bear no relation to the rates of return in the private sector, interest rates, or any other measurable market phenomena." [82]

3. ***Philosophy 3: The appropriate interest rate is dictated by the opportunity cost of those investments foregone by private investors who pay taxes or purchase bonds***, that is, "the rate of return that would have been experienced on the private uses of funds that would be precluded by the financing of the public project (say, through taxes or bonds.)" [82]

4. ***Philosophy 4: The appropriate interest rate is dictated by the opportunity cost of those investments foregone by budget agencies due to budget constraints****.* This too is an opportunity cost philosophy, which considers an artificial interest rate reflecting the rates of return foregone on government projects due to insufficient funds.

5. ***Philosophy 5: The interest rate should match that paid by government for borrowed money****.* This approach is favored by many people, and is supported by the argument that government bonds are in direct competition with other investment opportunities available in the private sector.

No one of these different philosophies is universally applicable, and no consensus exists as to which is preferable. The last of the five approaches listed is recommended in this Guide, and is discussed more below.

The rate at which governments can borrow money (by means of Federal, State, or municipal bonds) is widely felt to be the appropriate interest rate for evaluating highway improvement projects. Since government bonds are considered to be risk free, any public project should promise a minimum return that is at least equal to the bond rate. Government bonds are presumed to be lower risk than private investments, according to the rationale that governments are better positioned to cope with risk than are private investors. This is not to say that unexpected failures do not sometimes occur on individual publicly funded projects. Nonetheless, the relevant question in selecting an appropriate interest rate for a publicly funded project is, given the large numbers of projects that governments undertake, what is the cost to the government of the capital needed to finance its program of projects.

The discount rates used by State DOTs in life-cycle cost analysis vary from 0 to 10 percent, with values between 3 and 5 percent being most typical, and an overall average of about 4 percent. A value of about 4 percent is consistent with the average difference, over the last 30 years or more, between the interest rate on 30-year U.S. Treasury bonds, and the rate of inflation.

Determine Monetary Agency Costs

Monetary agency costs are all those costs associated with the alternative that are incurred by the agency during the analysis period and can be expressed in monetary terms. These include initial rehabilitation design and construction costs, follow-up rehabilitation design and construction costs, annual maintenance costs, traffic control costs during construction work, and either demolition-and-removal costs or residual value of the pavement structure at the end of the analysis period.

Only those agency costs that are significantly different for the different alternatives need to be considered in the life-cycle cost analysis. Engineering and administration costs, for example, may be excluded if they are the same for all alternatives. Rehabilitation and maintenance costs depend not only on the types and quantities of materials and work items, but also on the traffic control plan (detours, lane closures, work hours, etc.) selected for each alternative.

Determine Monetary User Costs

User costs are all those costs associated with the alternative that are incurred by users of a roadway over the analysis period and can be expressed in monetary terms. The users are both the actual and the would-be users, that is, those who cannot use the roadway because of either a detour imposed by the highway agency or the users' self-imposed selection of an alternate route.

There are three main categories of user costs:

1. **vehicle operating costs**, e.g., costs related to consumption of fuel and oil, and wear on tires and other vehicle parts;

2. **delay costs**, due to reduced speeds and/or the use of alternate routes; and

3. **accident costs** (also called crash costs), i.e., damage to the user's vehicle and/or other vehicles and/or public or private property, as well as injury to the user and others.

The term **in-service user costs** is used here for those costs in all three categories which are incurred during the normal use of the roadway over the analysis period. The term **work zone user costs** is used here for the extra costs in all three categories which are incurred due to lane closures and other aspects of construction, maintenance, and rehabilitation work. Among the factors which influence work zone user costs are the work zone length, number and capacity of open lanes, duration and timing of lane closures, speed restrictions, and availability and capacity of alternate routes.[83]

In principle, any in-service user costs and/or work zone user costs that differ significantly among the alternatives being considered should be taken into account in the life-cycle cost analysis. In practice, this may be difficult to do, due to the limited available information on user cost components and their relationships to project-specific details.

It may also be undesirable to the agency to attempt to consider all user costs components in the analysis, or to weight them equally with agency costs. Particularly for high-volume facilities, estimated user costs may be so high as to mask any other significant cost differences among alternatives. This is undesirable because the agency costs are those for which the agency really must program its funds. On the other hand, it is not advisable to exclude all consideration of user costs from the selection of rehabilitation strategy alternatives. Failure to consider work zone user costs may lead in many cases to the selection of excessively short-lived rehabilitation alternatives. For example, it would not be good practice to recommend rehabilitation of a major freeway every five years. Without quantitative consideration of work zone user costs, however, it is difficult to preclude the selection of such a strategy.

Each agency must make its own decisions about which user cost components the agency (a) expects to differ significantly by rehabilitation strategy alternative, and (b) is capable of estimating reasonably well. Some of the issues involved in considering in-service and work zone user costs in rehabilitation strategy life-cycle cost analysis are discussed below. More detailed discussion of user costs, and mention of other references on the subject, are provided in the FHWA Interim Technical Bulletin, *Life-Cycle Cost Analysis in Pavement Design*.[83] Computer programs are also available for use in analyzing user costs for individual highway pavement projects. More sophisticated network-level models are generally needed to analyze the effects of work zones on user costs for high-volume urban expressways.

Vehicle Operating Costs

In-service vehicle operating costs are primarily a function of pavement serviceability, i.e., roughness. It is sometimes said that in-service vehicle operating costs can be eliminated from consideration in a life-cycle cost analysis of pavement project alternatives because they are essentially the same for all alternatives. However, this should be recognized as a simplification. It is closer to being true when all alternatives (a) provide the same initial serviceability level, and (b) reach minimum acceptable serviceability at the same time. Even in this scenario, different serviceability trends may be associated with different alternatives (as illustrated earlier in Figure 21). The first difficulty then lies in being able to reliably predict these different serviceability trends.

When the different alternatives provide (a) different initial serviceability levels and/or (b) different performance periods (as illustrated earlier in Figure 22), then clearly they will have different serviceability levels at any given time, and thus have associated with them different in-service vehicle operating costs. Again, the first difficulty lies in being able to reliably predict these different serviceability trends.

In both scenarios, the second difficulty lies in estimating vehicle operating costs as a function of pavement serviceability level, particularly for the classes and traffic volumes of roadways in the United States and other highly developed countries. Some tools for modeling these costs do exist, such as the World Bank's Highway Design and Maintenance Standards Model (HDM-III),[84] the FHWA's Revised Highway Investment Analysis Package (HIAP-Revised),[85] the Texas

A&M Research Foundation's MicroBENCOST,[86] the AASHTO *Red Book*,[87] and others described in NCHRP Synthesis 269, *Road User and Mitigation Costs in Highway Pavement Projects*.[88]

Vehicle operating costs tend to be higher in work zones and detours, due to additional speed changes, stopping and starting, greater travel lengths, etc. Work zone vehicle operating costs may differ significantly for different rehabilitation alternatives, if different traffic control plans are associated with the different alternatives. Information on vehicle operating costs associated with stopping and starting, speed changes, and idling is provided in NCHRP Report 133, *Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects*.[89] This information is based on earlier work by Winfrey described in *Economic Analysis for Highways*.[90]

A detailed analysis of the potential impact of different traffic control options on work zone vehicle operating costs should consider not only average daily traffic volumes but also daily and hourly variations in traffic volume. Restricting the contractor's work hours, work days, and/or total project duration may reduce work zone vehicle operating costs but increase agency costs.

Delay Costs

User delay costs, i.e., the value of time, are the subject of considerable debate. In general, user delay costs vary by vehicle class, trip type (urban or interurban) and trip purpose (business or personal).

In-service user delay costs are primarily a function of demand for use of the roadway with respect to roadway capacity. Neither of these is likely to vary depending on the rehabilitation strategy alternative selected for a given roadway. Work zone user delay costs, however, may be significantly different for different rehabilitation alternatives, depending on the traffic control plans associated with the alternatives. Again, a detailed analysis of the potential impact of different traffic control options on work zone delay costs should consider not only average daily traffic volumes but also daily and hourly variations in traffic volume. Restricting the contractor's work hours, work days, and/or total project duration may reduce work zone user delay costs but increase agency costs.

Accident Costs

In-service accident rates for different roadway functional classes and accident types (fatal, nonfatal) are fairly well known. Accident costs are calculated by multiplying the unit cost per accident type, the crash rate per vehicle-miles traveled, and the vehicle-miles traveled (traffic per time period of interest multiplied by project length).

Limited data on accident rates in work zones suggest that they are about three times higher than the in-service accident rates. However, data are lacking on the relationship between work zone accident rates and traffic control details, such as lane narrowing, use of cones or other barriers, crossovers, etc. Data are also lacking on daytime versus nighttime work zone accident rates. Work zone accident costs are calculated by multiplying the differential work zone accident rate (e.g., about three) by the in-service accident costs that would be expected (from the above calculation) for the length and duration of the work zone.

If in-service accident costs depend primarily on the functional class of the roadway, then they should not be expected to differ for different rehabilitation alternatives and thus do not need to be considered in a life-cycle cost analysis of rehabilitation alternatives.

Work zone accident costs, on the other hand, may differ significantly for different rehabilitation alternatives if different traffic control plans are associated with the different alternatives. A detailed analysis of the potential impact of different traffic control options on work zone accident costs should consider not only average daily traffic volumes but also daily and hourly variations in traffic volume. Restricting the contractor’s work hours, work days, and/or total project duration may reduce work zone accident costs but increase agency costs.

Determine Other Monetary Costs

Other monetary costs are all those which are incurred by parties other than the agency or the users of the roadway, and which can be expressed in monetary terms. Among those others who might incur costs because of a rehabilitation project are, for example, the owners of properties and businesses adjacent to or near the route under study. Another example would be the municipalities whose sales tax receipts might be reduced during the period that the nearby businesses are adversely affected by the rehabilitation work.

Determine Residual Value

The residual value, also called the **salvage value**, of an alternative is the value that can be attributed to the alternative at the end of the analysis period. Residual value should be taken into account in comparing pavement rehabilitation strategy alternatives whenever the alternatives are expected to have significantly different residual values at the end of the analysis period.

How one defines the value of an in-service pavement determines how much residual value will be attributed to each alternative. The general definition of residual value is the value that the item would have in the marketplace. In the case of an in-service pavement, defining residual value requires giving some thought to what is the marketplace, i.e., what does the agency realistically intend to do with the pavement structure at the end of the analysis period.

One option is to recycle the pavement materials for use in reconstruction of the pavement and/or other road construction projects. The residual value in this case is the monetary value of the recycled materials minus the costs of removal and recycling. The residual value of the pavement structure as recycled materials may or may not be different for the different rehabilitation alternatives. Consider, for example, a pavement that is expected to require demolition and reconstruction at the end of the analysis period. If the agency typically recycles pavement materials, then the net value of the recycled materials is an appropriate measure of the residual value. However, if the agency does not plan to recycle the materials, there is no residual value, only a cost for demolition and removal. This cost is likely to be about the same for all alternatives.

An example of this approach to estimating salvage value is that used by the Arizona DOT.[73] The salvage value as a percent of the initial cost is determined as a function of the probability of the highway being rebuilt at the end of the analysis period (typically 50 percent likely at 35 years, in Arizona), the initial cost, the rehabilitation cost, the thicknesses of original pavement and any overlays, the worth of recycled asphalt concrete, and the removal cost. Salvage values calculated in Arizona using this method typically range from 31 to 39 percent of the initial cost for asphalt pavements, -2 to 27 percent for bare concrete pavements, and 22 to 39 percent for concrete pavements with asphalt overlays.

The residual value of a pavement that is likely to be rehabilitated at the end of the analysis period, rather than demolished, should be based on its contribution to the structural capacity of the rehabilitated pavement structure.

The FHWA's Interim Technical Bulletin on life-cycle cost analysis[83] recommends that the residual value be determined as the portion of the cost of the last rehabilitation equal to the portion of the remaining life of the last rehabilitation. For example, if an overlay with a predicted life of 12 years is placed 8 years before the end of the analysis period, it has a remaining life of 4 years at the end of the analysis period, so the residual value would be defined as 33 percent (4/12) of the cost of the overlay.

However, this method of defining residual value attributes worth only to the last rehabilitation application, rather than to the pavement structure as a whole. It may also have the undesired consequence of attributing greater worth to a pavement design or rehabilitation strategy alternative that performs poorly and requires frequent follow-up rehabilitation than to an alternative that performs better and requires less frequent rehabilitation.

Similarly, the serviceability level at the end of the analysis period is not necessarily a good indicator of residual value. A pavement with little remaining structural capacity could have a high serviceability level if it were overlaid near the end of the analysis period. However, that pavement may have little to contribute to the structural capacity of another rehabilitation.

When all alternatives are predicted to reach minimum acceptable condition at the end of the analysis period (and thus require rehabilitation at that time), another option for defining residual value is to determine what contribution the existing pavement structure will make to the structural capacity of the rehabilitated pavement structure. The residual value could be quantified as the portion of the rehabilitation cost that is reduced by the contribution of the existing pavement structure.

When one or more alternatives are predicted to reach minimum acceptable condition beyond the end of the analysis period, the residual values could be defined in terms of how long each alternative delays the next required rehabilitation. The residual value could be quantified as the difference between the cost of rehabilitation if performed at the end of the analysis period and the same cost of rehabilitation if deferred (i.e., discounted) some years into the future.

Thus, an alternative with more remaining structural capacity at the end of the analysis period would yield a larger difference between immediate and deferred rehabilitation costs, and therefore a higher salvage value. For this method of defining residual value to be appropriate, the same rehabilitation (and its cost) must be assumed for all of the alternatives, so that the only difference which the residual value reflects is the difference in remaining life, i.e., time to rehabilitation.

Whatever way residual value is defined for rehabilitation strategy alternatives, it must be defined the same way for all alternatives, and should reflect what the agency realistically expects to do with the pavement structure at the end of the analysis period. Only when the residual values as defined by the agency differ significantly among the rehabilitation strategy alternatives do they need to be included in the life-cycle cost analysis.

Compare Strategies by Common Economic Measure

Alternatives considered in a life-cycle cost analysis must be compared using a common measure of economic worth. The economic worth of an investment may be measured in a number of ways, including:

- **Present worth method**: the conversion of all cash flows, using a discount rate, to an equivalent single sum at time zero.
- **Annual worth method** (also called **equivalent uniform annual cost method**): the conversion of all cash flows, using a discount rate, to an equivalent uniform annual series of cash flows over the analysis period.
- **Future worth method**: the conversion of all cash flows, using a discount rate, to an equivalent single sum at the end of the analysis period.
- **Internal rate of return method** (also called **discounted cash flow method**): calculation of a discount rate for each alternative such that the values (e.g., present worths) of all alternatives are equal.
- **External rate of return method**: an alternative to the internal rate of return method which circumvents the possibility of a nonunique solution for the rate of return of an alternative (which may occur if there is more than one sign change in the sequence of cash flows).
- **Savings/investment ratio method** (also called **benefit-cost ratio method**): calculation of the ratio of the value (e.g., present worth) of positive cash flows to the value (e.g., present worth) of negative cash flows.

- **Payback period method**: determination of the length of time required to recover the initial investment based on a zero interest rate. It is important to note that this method is not equivalent to the preceding methods. That is, all of the preceding methods will yield the same decision regarding the relative desirability of the alternatives, but the payback period method in many cases will not.

- **Capitalized worth method**: the determination of the present worth of an amount that if invested would pay out a specified annual amount in perpetuity, e.g., for such long-term investments as the establishment of an endowment fund, or the perpetual maintenance of a bridge, dam, forest, or similar project.

The methods most commonly used to evaluate investment alternatives such as pavement rehabilitation strategies are present worth, annual worth (equivalent uniform annual cost), and internal rate of return. If quantifiable nonmonetary benefits are being considered as well, the alternatives can be compared in terms of their ratios of benefits to costs (either present worths or equivalent uniform annual costs), with higher benefit/cost ratios indicating preferred strategies. Consideration of nonmonetary cost and benefits is addressed in the next chapter of this report.

Present Worth

Present worth expresses all costs and benefits over the analysis period in terms of their equivalent value in the initial year of the analysis period. All costs incurred in the initial year (e.g., initial rehabilitation construction costs) are expressed in terms of their actual present value. All future costs (e.g., follow-up rehabilitation) and future benefits (e.g., residual value at the end of the analysis period) are discounted to their equivalent present values.

The general formula for the present value ($P) of a one-time future cost or benefit ($F) is the following:

$$\$P = \$F \times \left(\frac{1}{(1+i)^n} \right) \qquad \text{(Equation 7)}$$

where i = discount rate

n = year in which one-time cost or benefit occurs

Cost Example 1.

Consider a rehabilitation strategy alternative, such as an overlay, which has an initial rehabilitation construction cost, in year 1 of the analysis period, of $450,000 per four-lane mile. This overlay is expected to require a follow-up rehabilitation, such as a second overlay, in year 15 of the analysis period. Suppose the cost of this second overlay in current dollars would be $350,000 per four-lane mile. The present worth of the second overlay, assuming a discount rate of 4 percent, is calculated as follows:

$$\$P = \$350{,}000 \times \left(\frac{1}{(1 + 0.04)^{15}} \right) = \$350{,}000 \times 0.55527 = \$194{,}343$$

Considering, for the purpose of this example, only the initial rehabilitation construction cost and the follow-up rehabilitation cost, the present worth of this alternative is $450,000 + $194,343 = $644,343 per four-lane mile.

Costs which are expected to accrue annually (e.g., routine maintenance costs) can also be expressed in terms of their present worth. Such costs should be taken into consideration in the life-cycle cost analysis whenever they are expected to differ significantly for the different alternatives.

The general formula for the present worth ($P) of an annual future cost ($A) is the following:

$$\$P = \$A \times \left(\frac{(1 + i)^N - 1}{i (1 + i)^N} \right) \qquad \text{(Equation 8)}$$

where i = discount rate

N = number of years in the analysis period

Cost Example 2.

Suppose that the overlay alternative described in the previous example is expected to have annual maintenance costs of $2,500 per four-lane mile. The present worth of an annual maintenance cost of $2,500 per four-lane mile, over a 25-year analysis period, assuming a 4 percent discount rate, is calculated as follows:

$$\$P = \$2{,}500 \times \frac{(1 + 0.04)^{25} - 1}{0.04\,(1 + 0.04)^{25}} = \$2{,}500 \times 15.621 = \$39{,}052$$

The present worth of the overlay alternative over the 25-year analysis period, considering initial rehabilitation, follow-up rehabilitation, and annual maintenance costs, is therefore $450,000 + 194,343 + 39,052 = $683,395.

The conversion of future annual costs which are not uniform requires more than one step, requiring: (1) identification, if possible, of subperiods during which the annual costs are uniform, (2) converting these uniform annual costs to present worths in the beginning years of the subperiods, and (3) converting these present worths in given future years to equivalent present worths in the first year of the analysis period.

For example, suppose an additional uniform annual maintenance cost is expected to be incurred starting in year 15 of the 25-year analysis period. The present worth of these annual maintenance expenditures incurred between years 15 and 25 would be calculated by first using Equation 8 to convert the annual maintenance costs in years 15 to 25 to an equivalent present worth in year 15 (N = 10), and then using Equation 7 to convert this equivalent present worth in year 15 to an equivalent present worth in year 1 (N = 15).

Equivalent Uniform Annual Cost

Equivalent uniform annual cost expresses all costs over the analysis period in terms of an equivalent annual value that is the same for every year of the analysis period. Costs incurred in the initial year (e.g., initial rehabilitation construction costs) and one-time future costs (e.g., follow-up rehabilitation) and benefits (e.g., residual value at the end of the analysis period) can be expressed in terms of their equivalent uniform annual costs.

The general formula for the annual value ($A) of a cost in the initial year ($P) is the following:

$$\$A = \$P \times \left(\frac{i\,(1 + i)^N}{(1 + i)^N - 1} \right) \qquad \text{(Equation 9)}$$

where i = discount rate
N = number of years in the analysis period

Cost Example 3.

The equivalent uniform annual cost of the initial construction of the overlay alternative described in the previous examples, over a 25-year analysis period, assuming a 4 percent discount rate, is calculated as follows:

$$\$A = \$450{,}000 \times \frac{0.04\,(1 + 0.04)^{25}}{(1 + 0.04)^{25} - 1} = \$450{,}000 \times 0.06401 = \$28{,}845$$

To express a one-time future cost (e.g., follow-up rehabilitation) or benefit (e.g., salvage value) in terms of its equivalent uniform annual cost over the analysis period, it must first be converted to it equivalent present worth, using Equation 7, and then converted to its equivalent uniform annual cost, using Equation 9.

Cost Example 4.

For the overlay alternative described in the previous examples, a follow-up overlay was expected in year 15. The present worth of this follow up overlay in year 15 was $P = 194,343 (from Cost Example 1). The equivalent uniform annual cost of this follow-up overlay, over a 25-year analysis period, assuming a 4 percent discount rate, is calculated as follows:

$$\$A = \$194{,}343 \times \frac{0.04\,(1 + 0.04)^{25}}{(1 + 0.04)^{25} - 1} = \$194{,}343 \times 0.06401 = \$12{,}457$$

Annual costs that are uniform throughout the analysis period obviously require no conversion before being added to other equivalent uniform annual costs. Annual costs that are not uniform over the analysis period (e.g., annual maintenance costs forecasted for some subperiod within the analysis period) must be: (1) converted to present worth in the first year of the subperiod, using Equation 8; then (2) converted to present worth in the first year of the analysis period, using Equation 7; and finally (3) converted to equivalent uniform annual cost over the entire analysis period, using Equation 9.

Internal Rate of Return

The internal rate of return method can be applied to costs and benefits expressed in terms of present worth or equivalent uniform annual cost, although the more typical approach is to use present worth. When both costs and benefits are considered in a life-cycle cost analysis, and when both are expressed in the same monetary terms, the alternatives may be compared on the basis of the internal rate of return, which is the discount rate at which the costs equal benefits. Iteration is required to solve for the internal rate of return, but this can be done easily using a computer spreadsheet program.

The internal rate of return method may also be applied to an analysis of only costs or only benefits, by solving for the discount rate for each alternative which makes the present worths (or equivalent uniform annual costs) of all alternatives equal. When the internal rate of return method is applied to analysis of investment alternatives, with the goal of maximizing the net return on the investment, the preferred alternative is the one that has the highest internal rate of return. Conversely, when the internal rate of return method is applied to analysis of cost proposals, with the goal of identifying the alternative with the lowest present worth or equivalent uniform annual cost, as is the case in comparison of rehabilitation strategy alternatives, the preferred alternative is that which has the lowest internal rate of return.

For example, suppose the overlay alternative described earlier were compared against another rehabilitation alternative. For a common discount rate of 4 percent, the most cost-effective alternative of the two would be that which yielded the lower present worth (or equivalent uniform annual cost). Using the internal rate of return method of comparison, the second alternative would be more cost-effective than the first if it were found to yield the same present worth (or equivalent uniform annual cost) as the first alternative, at a discount rate less than 4 percent. This would suggest that the second alternative can achieve the same result for a lower cost of capital, and is thus the preferred alternative. On the other hand, if the two alternatives have the same present worth (or equivalent uniform annual cost) when the discount rate used for the second alternative is greater than 4 percent, then the first alternative is the preferred one.

The ranking of alternatives in practically all situations will be identical whether evaluated on the basis of present worth, equivalent annual uniform cost, or internal rate of return. The only exception to this is when more than one sign change occurs in the sequence of cash flows. This may lead to multiple solutions by the internal rate of return method. In this situation the external rate of return method should be used instead.

Weigh Agency Costs Versus User Costs

An agency's concerns about how much emphasis to place on user costs can be addressed by weighting agency costs and user costs as the agency deems appropriate, as suggested by the following equation:

$$PW_i = \Sigma \; a_1 (AC)_i + a_2 (UC)_i \qquad \text{(Equation 10)}$$

Where Pw_i = total present worth of rehabilitation strategy i

a_1 = weighting factor for all agency costs

a_2 = weighting factor for all user costs

AC_i = present worth of agency costs over analysis period for strategy i

UC_i = present worth of user costs over analysis period for strategy i

Assess Sensitivity of Life-Cycle Cost Analysis to Key Parameters

A thorough life-cycle cost analysis of rehabilitation strategies should also include a sensitivity analysis of the parameters that most influence the relative cost-effectiveness of different alternatives. Among the more sensitive factors are:

- **The analysis period and performance period:** Both of these parameters have a considerable influence on the rehabilitation strategy (the combination of treatment types and timing) that is most cost-effective for a given pavement.

- **The predicted traffic over the design and analysis periods:** this has an important influence on both the performance of the initial rehabilitation design and the performance of all follow-up maintenance and rehabilitation treatments.

- **The initial investment:** the initial costs of rehabilitation construction typically constitute well over 50 percent of the total present worth of a rehabilitation strategy alternative. There is uncertainty associated with the initial rehabilitation construction costs, because, for one reason, the life-cycle cost analysis of rehabilitation strategy alternatives is typically conducted well in advance of the solicitation for construction bids.

- **The discount rate:** A lower discount gives more weight (more importance) to future costs, while a higher discount rate gives less weight to future costs. That is, a higher discount rate favors strategies with costs deferred into the future, while a lower discount rate does not demonstrate as great a benefit to deferring costs into the future. Thus, the sensitivity of the life-cycle cost analysis results to the discount rate used is of more concern when comparing alternatives with very different future costs.

- **The timing of follow-up maintenance and rehabilitation activities:** These presumably are based on predicted pavement performance and thus have associated with them considerable uncertainty.

- **The quantities associated with initial and follow-up maintenance and rehabilitation activities:** These too are based on predicted pavement performance and thus have considerable uncertainty associated with them.

Many good basic references on engineering economics are available which cover, among other subjects, the essentials of identifying alternatives, calculating the time value of money, selecting an appropriate analysis period, and methods for comparing alternatives by a common economic measure. *Essentials of Engineering Economics*, by Riggs and West [91] is one of several such references. An excellent reference is *Principles of Engineering Economic Analysis*, by White, Case, Pratt, and Agee.[80]

Among the references which address the application of economic analysis to highways are: Winfrey´s *Economic Analysis for Highways,*[90] Adler´s *Economic Appraisal of Transport Projects,*[92] the *1993 AASHTO Guide,*[1] the Federal Highway Administration´s Interim Technical Bulletin on *Life-Cycle Cost Analysis in Pavement Design,*[93] the American Concrete Pavement Association´s Technical Bulletin on *Project Life-Cycle Cost Analysis,*[94] and NCHRP Synthesis 122 on *Life-Cycle Cost Analysis of Pavements.*[95] These references are focused more on comparing highway investment alternatives and/or comparing pavement design alternatives than on comparing pavement rehabilitation strategy alternatives.

Chapter 7

Guidelines for Consideration of Monetary and Nonmonetary Factors in Rehabilitation Strategy Selection

The purpose of this final step is to select one of the rehabilitation strategy alternatives developed and evaluated previously. The strategy selected may simply be the strategy identified in the life-cycle cost analysis as having the lowest total monetary cost. However, pavement rehabilitation projects may have costs and/or benefits which cannot be measured in monetary terms, but which can affect the decision as to which rehabilitation strategy alternative is selected. Among the nonmonetary factors which might influence the selection are the following:

- Geometric restrictions.
- Construction duration.
- Environmental impact (e.g., contamination generated during construction work).
- Conservation of natural resources.
- Agency's experience with the use of the rehabilitation techniques involved.
- Traffic safety during construction.
- Worker safety during construction.
- Contractors' experience with the rehabilitation techniques involved.
- Availability of needed equipment and materials.
- Competition among providers of materials.
- Stimulation of local industry.
- Political concerns.

Rehabilitation strategy alternatives involving substantial overlay thicknesses often face geometric restrictions, especially in urban areas. One State DOT recommends a preliminary investigation to determine the amount of pavement removal and undercutting necessary to meet at-grade bridges and provide clearance under overhead bridges. If the amount of pavement removal necessary exceeds about 40 percent, assuming none of the bridges are jacked, the alternative is eliminated from consideration.[68]

One approach to considering monetary and nonmonetary factors together is by assigning them weights indicative of their relative importance to the agency. This is the same concept as was described earlier for weighting agency costs versus user costs. The consideration of monetary and nonmonetary factors together in the rehabilitation strategy selection process is useful when some benefits and/or costs, while technically monetary, are difficult to quantify monetarily because of lack of cost data.

For example, rather than attempting to express in-service vehicle operating costs monetarily, an agency may wish to express the benefits to the users of a smoother pavement in some other way. The agency could assume or predict the serviceability trend for each alternative over the analysis period (e.g., using the 1993 AASHTO design methodology). Two quantitative measures of the benefit that each strategy's performance provides to the users, both fairly easy to calculate, are the area under the serviceability curve over the analysis period and the average serviceability over the analysis period. Indeed, the performance of a pavement design or rehabilitation strategy is often said to be represented by the area under its serviceability curve, and the average serviceability is just a simpler calculation that expresses the same performance. As illustrated earlier, different rehabilitation strategies may have significantly different serviceability trends, and thus have significantly different benefits to the users.

If the benefits to the users of smoother pavements are not considered in the rehabilitation strategy selection process, neither monetarily (in terms of in-service vehicle operating costs), nor nonmonetarily (in terms of the area under the serviceability curve), the life-cycle cost analysis will indicate that the preferred strategies are those which achieve and maintain the required minimum condition at the lowest cost to the agency. Rehabilitation treatments which permit more rapid declines in serviceability also usually have higher maintenance needs, so both user costs and agency costs may be underestimated if serviceability levels and associated maintenance costs are not considered in the rehabilitation strategy selection process.

Chapter 8

Summary and Conclusions

This *Guide for Selection of Pavement Rehabilitation Strategies* provides a step-by-step process and practical guidance for project-level evaluation and rehabilitation strategy selection for in-service pavements. Pavement rehabilitation is defined for the purposes of this Guide as a structural or functional enhancement of a pavement which produces a substantial extension in service life, by substantially improving pavement condition and ride quality

A review of the pavement rehabilitation practices of State DOTs, and the literature available on pavement evaluation, rehabilitation techniques, and selection of rehabilitation strategies, was conducted for this project. Although all State DOT agencies are engaged in pavement rehabilitation, fairly few of them have any more than the most simple and general guidelines for selection of rehabilitation strategies. The rehabilitation strategy selection procedures used by the various highway agencies differ in their details, but typically consist of the following principal activities:

- **Data collection:** Gathering all of the information necessary to conduct an evaluation of the pavement's present condition and its rehabilitation needs.
- **Pavement evaluation:** Assessing the current condition of the pavement, identify the key types of deterioration present, identify deficiencies that must be addressed by rehabilitation, and identify uniform sections for rehabilitation and design over the project length.
- **Selection of rehabilitation techniques:** Identifying candidate rehabilitation techniques which are best suited to the correction of existing distress and achievement of desired improvements in the structural capacity, functional adequacy, and drainage adequacy of the pavement.
- **Formation of rehabilitation strategies:** Combining individual rehabilitation techniques into one or more rehabilitation strategy alternatives, developed in sufficient detail that the performance and costs of each may be confidently estimated.

- **Life-cycle cost analysis:** Comparing the monetary costs and benefits of the different rehabilitation strategy alternatives over a common analysis period.

- **Selection of rehabilitation strategy:** Considering monetary factors and nonmonetary factors together in selecting one pavement rehabilitation strategy from among the alternatives considered.

Data Collection

Guidelines are provided for collecting the data needed to make informed decisions in selecting rehabilitation strategies. The data collection begins with defining the roadway section to be rehabilitated and obtaining inventory information on the pavement layers, pavement materials, subgrade, subdrainage, and roadway geometry. Traffic data for the section are needed to characterizing characterize present and future truck traffic loadings. Field data collection may include distress data collection surveys; nondestructive deflection testing, materials sampling for laboratory testing, profile and roughness measurement, friction measurement, drainage inspection, and other nondestructive testing (e.g., ground-penetrating radar).

Pavement Evaluation

Rehabilitation of a pavement is most likely to be successful if it is selected on the basis of knowledge of the types of distresses occurring in the pavement, and understanding of the causes for those distresses. This knowledge and understanding is necessary to identify rehabilitation methods that effectively repair and prevent recurrence of the distress. Guidelines are provided for using the information obtained in the distress survey to diagnose the causes of the distresses seen. Many distresses have more than one possible cause. It is important to carefully study the distresses observed to determine their most probable causes.

Structural evaluation involves examining the collected distress, deflection, materials, soils, and drainage information for the purposes of assessing the current structural condition of the pavement and the remaining life of the pavement. Structural evaluation for asphalt pavements may be accomplished using condition data only, distress data only, condition plus deflection data, or traffic data only (although there are significant limitations to this last approach). Structural evaluation for concrete and asphalt-overlaid concrete pavements is accomplished primarily using condition-based methods, although deflection data may be very useful in this process. Trigger values are suggested for key condition levels (asphalt pavement cracking, concrete pavement cracking and faulting, composite pavement reflection crack deterioration) at which a pavement is generally considered to need a structural improvement.

Similarly, trigger values are suggested for key condition levels (asphalt pavement rutting, concrete pavement faulting, serviceability in all pavement types) at which a pavement is generally considered to need a functional improvement.

The evaluation of drainage adequacy in existing pavements has in the past been largely a cursory exercise, limited to observations on whether or not water seemed to be flowing from drainage outlets. Some past and current research advocates a more objective assessment of (a) whether or not the climate is such that the expected inflow into the pavement is greater than the drainage capacity of the pavement base and natural subgrade, and (b) whether or not the subdrainage features, if present, are adequate to accommodate any excess inflow which might occur.

Selection of Rehabilitation Techniques

Although just about any rehabilitation technique can be applied at any time, the goal of rehabilitation strategy selection is to identify the techniques that are best suited to the types of distress present. These are the techniques that are most likely to produce satisfactory performance and cost-effectiveness in a rehabilitation treatment.

Guidelines are provided to identify the individual rehabiitation techniques that are believed to be best suited to each of the distresses commonly seen in each of the three pavement types considered (asphalt, concrete, and asphalt-overlaid concrete). Some users of this Guide may disagree with some of the details of these guidelines – perhaps because those users have had poor experience with one of the techniques indicated, or have had good experience with a technique that is not indicated. These guidelines may of course be customized to the experience of the individual users.

This Guide includes a summary of each of the following pavement rehabilitation techniques:

Asphalt Pavement Rehabilitation Techniques

Asphalt patching
Cold milling
Hot in-place recycling
Cold-in place recyling
Asphalt overlay
Concrete overlay

Concrete Pavement Rehabilitation Techniques

Full-depth repair and slab replacement
Partial-depth repair
Undersealing (slab stabilization)
Load transfer restoration
Diamond grinding
Asphalt overlay
Asphalt or concrete overlay of fractured concrete slab
Bonded concrete overlay
Unbonded concrete overlay

Each summary addresses appropriate use of the technique, limitations (for some techniques), concurrent work, materials, design (for some techniques), construction, performance, and references. Additional considerations for the application of some of these techniques to existing asphalt-overlaid concrete pavements are mentioned where appropriate.

Formation of Rehabilitation Strategies

The individual techniques selected to address the distresses and structural and functional deficiencies observed must be combined into one or more feasible rehabilitation strategy alternatives, developed in sufficient detail that their performance and costs may be confidently estimated.

Combining individual techniques into one or more rehabilitation strategies requires recognizing when some techniques supercede others, and when some techniques are incompatible with others. The steps in the process of combining rehabilitation techniques into strategies for any pavement are the following:

1. Select a structural improvement to correct a structural deficiency, if present;

2. Select a functional improvement to correct a functional deficiency, if present and not corrected by a structural improvement,

3. Select additional repair/restoration techniques to correct distresses not corrected by a structural or functionl improvement, and

4. Consider a drainage improvement to correct a drainage deficiency, if present.

Tables are provided in the Guide which list all possible feasible combinations of structural, functional, and repair techniques for each pavement types. The designer may select all of those combinations which address the conditions of the particular pavement being addressed, or may select some subset of the combinations. In addition, there are multiple variations possible with respect to many of the combinations. For example, a strategy which involves asphalt overlay with preoverlay full-depth asphalt repair may have many permutations, depending on the amounts of existing distress to be repaired and the corresponding overlay thicknesses required.

The options for making drainage improvements to in-service pavements are presented with the caveat that it is difficult to predict what effects, either positive or negative, drainage improvement efforts may have on the performance of a rehabilitated pavement. Very little research is available to demonstrate how retroffitted subdrainage influences in-service pavement performance.

Rehabilitation strategies involving overlay require that the thickness of the overlay be determined. This Guide is structured in such a way that overlay thicknesses may be obtained from any of several different methods, including the 1993 AASHTO method, the Asphalt Institute method, the Portland Cement Association method, the Corps of Engineers method, or any of several State DOT methods. It is not within the scope of this Guide to provide details about the use of these overlay design procedures, nor to provide performance prediction models for other types of rehabilitation.

The most challenging aspect of rehabilitation strategy selection is the prediction of the performance of the different rehabilitation strategies being considered. Rehabilitation performance prediction involves, as a minimum, predicting the time (either in years or accumulated axle loadings) at which each rehabilitation strategy will reach a condition requiring follow-up rehabilitation. Rehabilitation performance prediction may also involve predicting the shape of the performance curve over its life.

In this Guide, general estimates have been provided for the service life ranges for several different types of rehabilitation strategies. These ranges are general estimates only, expressed in years, not including consideration of traffic level. The ranges are intended to represent the "conventional wisdom" about the service lives that may reasonably be expected of the different rehabilitation techniques. Service life estimates used by specific State DOTs or other agencies are cited where possible for comparison.

The performance of a given rehabilitation strategy is very difficult to predict. Rehabilitation performance is particularly sensitive to the following factors:

- Appropriateness of use of the technique for the type of distress present, and for the point in the structural life of the pavement at which the rehabilitation is applied;
- The combination of techniques applied;
- For overlay and reconstruction techniques, the structural design (thickness, joints, etc.)
- The extent, type, and construction quality of preoverlay repairs;
- The materials used;
- The construction quality of the rehabilitation;
- The truck traffic level;
- The climate; and
- The level of routine maintenance applied.

The sad fact is that the state of the art of performance prediction for rehabilitated pavements currently lags far behind that of performance prediction for new asphalt and concrete pavements.

Life-Cycle Cost Analysis

A life-cycle cost analysis of pavement rehabilitation strategy alternatives, when done correctly, permits the identification of the strategy which yields the best value, by providing the desired performance at the lowest cost over the analysis period. Ideally, a comprehensive life-cycle cost analysis would consider quantitatively all of the costs incurred by both the agency and the users over the analysis period. However, some of these costs are difficult to quantify, necessitating some simplifications to the life-cycle cost analysis. Furthermore, differences of opinion exist about how to quantify some inputs to life-cycle cost analysis, and about how to define the inputs to the analysis. Each agency must make its own decisions about which costs (a) the agency expects to differ significantly among rehabilitation strategy alternatives, and (b) the agency is capable of estimating reasonably well.

This Guide provides detailed guidelines for life-cycle cost analysis of pavement rehabilitation strategy alternatives, including selection of an appropriate analysis period, selection of an appropriate discount rate, determination of monetary agency costs, determination of monetary user costs, determining other monetary costs, and assigning residual value to the alternatives. The different philosophies for defining residual value are discussed. It is not, however, within the scope of this Guide to provide default unit costs for the different items which may enter into a life-cycle cost analysis.

There are two general categories of user costs, those in-service user costs incurred during the normal use of the roadway over the analysis period, and those work zone user costs incurred due to lane closures and other aspects of construction, maintenance, and rehabilitation work. Within each of these two categories, there are three types of user costs: vehicle operating costs, delay costs, and accident costs. It may not be feasible or desirable for an agency to attempt to consider all of these different types of user costs quantitatively in the selection of pavement rehabilitation strategies. On the other hand, failure to consider any user costs (e.g., work zone delay costs) may lead in many cases to the selection of excessively short-lived rehabilitation alternatives. Suggestions are given for how agency costs and user costs may be considered together, using weighting factors that reflect the relative importance of these cost factors to the agency.

The Guide defines several different economic measures that may be used to compare strategies, and provides detailed guidance on the use of three of these measures: present worth, annual worth (also called equivalent uniform annual cost), and internal rate of return.

Selection of Rehabilitation Strategy

The rehabilitation strategy which is ultimately selected may simply be that which was found in the life-cycle cost analysis to be most cost-effective. However, in many cases, an agency will wish to weigh the cost analysis results with other decision factors that cannot be expressed in monetary terms. Here too, relative weights may be assigned to various monetary and nonmonetary factors, according to the relative importance of those factors to the agency.

The conduct of this project has demonstrated clearly that, despite the enormous amount of funding dedicated to pavement rehabilitation in the United States every year, the pavement field’s ability to predict the performance of different rehabilitation techniques – primarily as a function of their time of application, preoverlay repair, and thickness design – remains very limited. A great deal has been written about how rehabilitation techniques should be constructed and what materials should be used, but relatively little useful research has been done into how long and how well these different rehabilitation techniques perform. This Guide provides the designer with a step-by-step process for project-level evaluation of pavements in need of rehabilitation, selection of rehabilitation techniques believed to be appropriate, and formation of rehabilitation strategies expected to be feasible and cost-effective. As the pavement field's ability to predict rehabilitation performance improves, the process outlined in this Guide may be further refined and customized to the needs of individual State agencies.

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